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ELEMENTS

OF

PRACTICAL CONSTRUCTION,

FOR THE USE OF

STUDENTS IN ENGINEERING AND ARCHITECTURE.

PART I.

STRUCTURES IN DIRECT TENSION AND COMPRESSION.

With Atlas of Plates and numerous Woodcuts.

BY

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PREFACE.

THE following pages on the Elements of Practical Construction have been compiled for students in Engineering, on the same general plan as that adopted by the author in a work on Practical Hydraulics, which has been received with considerable favour by those for whose use it was intended.

The present work treats only of the resistance of materials to direct compression and tension, leaving to another volume the subjects of elasticity, indirect compression, and tension, transverse resistance and torsion, &c.

In treating of each material a Proposition is first given stating its average ultimate resistance; this is followed by experimental proofs; and then are given illustrations of the material so strained taken from completed and successful structures of eminent engineers.

The experiments and some of the examples now published in a collected form have been taken from the Proceedings and Transactions of learned Societies, which are not easy of reference, and cannot readily be purchased, and from Reports of Commissions, with their Appendices, all which are too bulky for the library of beginners in a profession which demands frequent change of residence. To give the chief

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points of these in a more portable form, with practical examples added, may, it is hoped, be of some use. The Provost and Senior Fellows of Trinity College, Dublin, have paid a part of the expenses of this work, and the author takes the present opportunity to thank them for their liberality.

ELEMENTS OF PRACTICAL CONSTRUCTION.

RESISTANCE OF MATERIALS.

1. An attentive examination of any engineering structure, or of any machine in motion, must lead the observer to the conclusion that the several component parts are resisting, in very different modes, the diverse pressures and strains known to be in action.

Four classes of these resistances are generally enumerated:—

First.—The resistance to direct extension; a force being in action tending to tear asunder the fibres or particles of the body. Tie-rods, tie-beams, the links of the main chains of a suspension bridge, the pump rods of pumping engines, ropes and chains, chain cables, rivets, and screw bolts, and the external plates of steam-engine boilers, &c., are instances of this description of resistance being brought into action.

Second.—The resistance to direct compression, a force being present tending to crush the particles or fibres. Pillars, piers of bridges, posts and struts, the shafts of columns, the main ribs of iron arches, and voussoirs of bridges of masonry, afford examples of this particular class of resistance to the external forces in action.

Third.—The resistance of a beam supported at one or both extremities to a force acting in a direction transverse to its length, and in fibrous material, as wrought iron and timber, perpendicular to the line of fibres. The girders of bridges, all beams, bresummers, and joists, and the beams of steam-engines, are instances of this, which is in fact, as will be shown, compounded of the two first-named modes of resistance, one part of the cross section of the beam, through the *indirect* action of the force applied, resisting a compressive, and the other a tensile force.

Fourth.—The resistance to forces of torsion which act so as to twist the fibres, as in crank axles, the shafting of machinery, the axles of water-wheels, capstans, &c. This, as well as the third kind of resistance, is compounded of the resistance to compressive and tensile forces; one portion of the body resisting the former, and another the latter force.

2. The ordinary double-acting steam-engine illustrates in one machine all the four classes of resistance named above. In the ascent of the piston, the steam forcing it upwards, we have the piston rod and the connecting rod resisting compression; in the descending stroke they are resisting a tensile force. In each case the main beam is resisting a transverse strain, and the crank axle resisting the torsion or twist arising from the power applied at the end of the crank arm, on the one hand, and the resisting load at the first wheel or band, on the other.

Many other complicated varieties in the mode of action of the resisting powers of bodies might be enumerated in addition to these four, such as that which takes place from the interior to the exterior surface of hydraulic presses, and of pieces of ordnance when fired, &c. &c.; and, so far as at the present time they have been investigated, they shall be mentioned hereafter.

The materials whose power of resistance it is, as far as the practice of engineering is concerned, most important to determine, are, Cast Iron, Wrought or Malleable Iron, and Timber, chiefly fir, as imported from the Baltic or America.

The properties of some other bodies, viewed in connexion with their resisting powers more or less essential in the art of construction, such as stone, brick, &c., are also given in this work.

DIRECT RESISTANCE OF CAST IRON, MALLEABLE IRON, AND TIMBER, TO TENSILE FORCES.

Cast Iron.—*The ultimate resistance of Cast Iron to a direct tensile force is about 7 tons per square inch, and is proportional to the area of transverse section in action.*

3. In experimenting on any material with a view to determine its resistance to a tensile force, it is necessary that the axis of the several pieces under trial coincide very truly with the line of direction of the external force applied to tear them asunder. The *form* of the specimen to be experimented upon, must therefore be carefully designed, having reference both to this essential condition as well as to the nature of the material under trial; the extremities being such as may be firmly held by the apparatus, and transmit the applied force truly through the centre of figure or axis of the specimen. Fig. 1 represents an isometrical projection of the form of the castings used in the experiments tabulated §§ 8, 9; at each end of the specimen, the mass of metal is considerably greater than in the centre, in order that the pieces may be nearly perfectly rigid at the extremities; for a length of about one foot in the middle the transverse section is cruciform, Fig. 2, and this part being made weaker than the ends, it was intended to be there torn asunder, by a force acting in the vertical through its centre of figure. The ends of these castings had eyes drilled through them, of a conical form, as shown at *a* and *b*, with a prominence in the central line shown in the longitudinal section in Fig. 3. The casting is shown Fig. 4, with the shackles attached above and below; the bolt passing through the eye bears upon the prominence mentioned above, effectually securing the desired central direction of the forces thus applied at the extremities of the specimen under trial.

4. Fig. 5 represents the apparatus used for tearing asunder the castings described above, and is equally applicable to specimens of any other material, whose direct resistance of cohesion is sought. It consists of a lever of the second order, attached to a strong frame, in which the ratio of the arms is 8 to 1, and as it is important that the applied force upon the specimens under trial should during the experiment be maintained truly vertical, and the lever horizontal, adjusting screws are placed at E and F, so that if the specimen should be elongated by the

strain, causing the lever to be depressed at C, it may again be brought to the horizontal line by bringing down the end A. These screws are also useful in placing specimens of different length in the machine.

5. The specimen of cast iron is shown in its particular position in the apparatus at D, the point D being that at which it is intended to be dissevered. The piece to be torn asunder is connected at one end by shackles with the lever at B, and at the other with a supporting cross timber by a screw at E, which admits of adjustment, so as to have the lever horizontal at every different length of specimen. The lever at the end A is sustained by a strong bolt AF passing through a balk of timber at the bottom, and tightened and adjusted at the point A by a screw F. The bolt acts upon the lever by means of an arris or knife-edge, and therefore presses only upon a mere line across it; as shown by the dotted lines. The same remark applies to the strap round the lever at B; the lever is sustained in the vertical plane by means of two pieces of timber connected together, represented by M, N; between these it moves easily, and can be sustained in any position, by wedges which rest upon the middle piece that extends up to that height. In order to show more completely all the detail, the main timber framing is supposed to be removed on the right-hand side, the plane being indicated by the dotted line X, Z, on Front Elevation.

6. Other forms of apparatus for testing the ultimate resistance of materials and proving bars of a particular design are described by Barlow, pp. 250, 259, who gives a detailed account of the Proving Machine in the Dockyard, Woolwich.—Report on Metal for Cannon by the American Ordnance Department, pp. 305, 315.—Mr. Provis on the Menai Bridge, pp. 34, 35 (vv. aa. No. 50, Library, T. C. D.)

7. Although experimenters on the ultimate resistance of materials have devoted so much attention to the accurate determination of the ultimate tensile powers of cast iron, the student must not suppose that it is ever applied in any structure as a rod or bar to resist tension. A design in which such an arrangement was found would be condemned by every practical engineer; the material best adapted for such a position and duty, and the various grounds on which that preference is founded, will soon be made apparent. The real value of the knowledge thus attained is this: that it enables us—when we have by experiment also decided its ultimate resistance to

compression—to determine the proper form of beams of cast iron; for as one portion of every beam is resisting tensile, and the other compressive forces, it is necessary to the proper distribution of the material in transverse section, that the ratio of its power of resistance to these different forces be known.

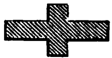
8. ULTIMATE RESISTANCE OF CAST IRON TO A TENSILE FORCE, ABOUT 7 TONS PER SQUARE INCH OF SECTION.

Experimental Proofs.

Table *showing the Tensile Strength of Cast Irons of various denominations in common use.*

(Compiled from Table I. Appendix A, Report of Commissioners on Application of Iron to Railway Structure, 1849.)

	Description of Iron.		Specific Gravity.	Area at D.	Breaking Weight in lbs.	Breaking weight, per Square inch of Section.
				sq. inches.		Tons.
I.	Low Moor Iron,	No. 1,	7.074	4.230	53718	5.667
II.	,, ,,	No. 2,	7.043	4.282	66174	6.901
III.	Clyde Iron,	No. 1,	7.051	4.286	69160	7.198
IV.	,, ,,	No. 2,	7.093	4.394	78216	7.949
V.	,, ,,	No. 3,	7.101	4.347	101958	10.477
VI.	Blaen avon Iron,	No. 1,	7.042	4.259	59304	6.222
VII.	,, ,,	No. 2,	7.113	4.314	72147	7.466
VIII.	,, ,,	,,	7.051	4.366	95718	6.380
IX.	Calder Iron,	No. 1,	7.025	4.360	59901	6.131
X.	Coltness Iron,	No. 3,	7.024	4.355	67219	6.820
XI.	Brymbo Iron,	No. 1,	7.071	4.368	62982	6.440
XII.	,, ,,	No. 3,	7.037	4.358	67611	6.923
XIII.	Bowling Iron,	No. 2,	6.989	4.406	59547	6.032
XIV.	Ystalavera Iron,	No. 2,	7.147	4.148	52080	5.602
XV.	,, ,,	,,	7.092	3.076	54395	7.791
XVI.	Ynis-cedwin Iron,	No. 1,	7.034	4.247	59248	6.228
XVII.	,, ,,	No. 2,	7.013	4.508	60144	5.959
XVIII.	Means, ...		7.059			6.834

The section of the castings was of this form , and the experiments were made in the manner explained in the description of the apparatus for ascertaining the tensile strength of cast iron, §§ 4, 5.

9. The figures in the several columns in the above Table are

averages taken from those given in the Report, in order to show the broad basis on which the results are founded; columns Nos. I. and X. are here added in full.

	1	2			3	4	5	
			Mean.	General Mean.				Mean.
I.	Low Moor Iron, No. 1,	7.133 7.088 7.045 7.029	7.111 7.037	7.074	4.301 4.144 4.154 4.336 4.214	64227 47059 51539 47955 57811	14933 11355 12407 11059 13718	12694 lbs. = 5.667 tons.
X.	Coltness Iron, No. 3,	7.070 7.013 7.034 6.977	7.042 7.006	7.024	4.502 4.264	78419 56019*	17418 13137	15278 lbs. = 6.82 tons.

* If the first result only had been taken, the strength would have been 7.776 tons, instead of 6.82 tons. A defect of ¼ inch in area nearly was observed.

10. In the Appendix are also given experiments on Mr. Morris Stirling's patented mixture of cast and wrought iron. That denominated second quality consisting of Calder No. 1, hot blast, mixed and melted, with about 20 per cent. of malleable iron scrap, gave a strength of 11.502 tons per square inch. That called third quality, composed of No. 1 hot blast, Staffordshire iron, from Ley's works, mixed and melted with about 15 per cent. of common malleable iron scrap, gave 10.474 tons per square inch.

11. Each of the numbers in column 2, §§ 8, 9, has been deduced from four experiments, two of them on portions cut from the thicker part of the casting torn asunder, and two from the thinner part: it was found that, with one exception (namely, line III.), the specific gravity of the former was greater than that of the latter,—the mean from all the irons being 7.082 for the specimen from the thicker portion, and 7.036 for that from the thinner.

"This difference may arise, either from the thinner parts being more stretched during the experiments than the thicker, and retaining in some degree their superior elongation, or else possibly from thinner castings in general being lighter than thicker ones of the same bulk, which would require trials upon thick and thin portions of castings which had not been experimented upon for the tensile strength.

"In timber and some other materials the strength has been found to be nearly as the specific gravity (Barlow, pp. 4 and 5); but this does not seem to be the case in cast iron; though the heavier irons, of the same kind, are usually stronger than the lighter ones."

An extended Table of the specific gravity of cast irons of British make has been given by Mr. Mallet (Trans. Royal Irish Academy, vol. xxiii. part 1), containing 59 classes of iron; the maximum density was 7.624, the minimum 6.809; forty-seven of them gave results below 7.2; all were deduced from equal pieces, cast one inch thick, and five inches square.

Tredgold gives the number 7.207; for approximate calculations 7¼ is often used.

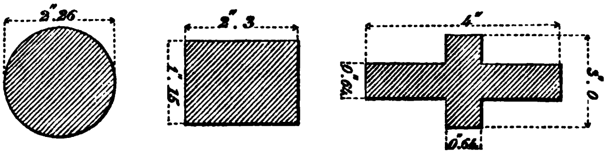
12. Other experiments on the ultimate tensile strength of cast iron by Mr. Hodgkinson give results in which the minimum resistance was 6 tons, and the maximum 9.755 tons per square inch; the mean of the whole set of experiments, twenty-five in number, being 16505 lbs., or 7.37 tons nearly. A series of ten careful experiments made in France gave a mean of 7.19 tons, the minimum being 5.409, and the maximum 9.080 tons per square inch. Thus we have:—

	Maximum.	Minimum.	Mean.
Experiments by Mr. Hodgkinson,	9.755	6.00	7.37
,, in Appendix A, Report,	10.477	5.667	6.834
,, undertaken in France,	9.080	5.409	7.19
The general Mean being, ...	. . .	. . .	7.131

EXPERIMENTS TO DETERMINE THE TENSILE STRENGTH IN DIFFERENT FORMS OF SECTION, AND TO ASCERTAIN WHETHER, WITH THE SAME AREA, THE FORM HAS ANY INFLUENCE ON THE STRENGTH.

13. In order to meet objections that had been raised against the results of these experiments on the ultimate resistance of cast iron, objections grounded on the peculiar form of the pieces subjected to the breaking strain,—Mr. Hodgkinson undertook a series of experiments on bars of the three several forms of cross section shown in these woodcuts, in which the transverse area was intended to be the same in each, namely,

4 square inches. They were, moreover, all run at the same time from the same melting of iron, so as to secure identity of



quality in each specimen. The results were such that, assuming the resistance of the cruciform section to be represented by 1000, that of the other forms was found to be 989½; these numbers being derived from a mean of 22 experiments undertaken with the special object of determining this point.

Hence it would appear that there is no essential difference in the ultimate resistance of cast iron arising from the form of the section only; the difference that does exist in favour of the cruciform section is chiefly attributable to the metal being harder in the thinner sections than the others, which had evidently less surface for the same quantity of iron.

EXPERIMENTS INTENDED TO DEMONSTRATE THE IMPORTANCE OF THE COINCIDENCE OF THE RESULTANT OF THE FORCES APPLIED WITH THE AXIS OF THE SPECIMEN.

14. The form of casting described above enables the experimenter to direct the force applied with greater certainty through the axis of the specimen. The great importance of this condition is shown by comparing the results of two sets of experiments made by Mr. Hodgkinson, in one of which the direction of the tensile force, that is, the resultant of all the parallel forces applied, passed truly through the axis or centre of figure of each piece, and the other along the side.

15. *Force up the centre or axis.*

Experiments.	Area of section in parts of an inch.	Breaking weight in tons.	Strength per square inch.
1	3.012	22.5	7.47 Tons.
2	2.97	21.0	7.07 Mean, 7.65 = 17136 lbs.
3	3.031	25.5	8.41
4	2.95	19.5	6.59 different quality of Iron.

16. *Force along the side.*

Experiments.	Area of section in parts of an inch.	Breaking weight in tons.	Strength per square inch.	
5	4.83	11.5	2.38	Mean, 2.62 tons.
6	4.815	13.75	2.855	

Thus it appears, that in the first series the average breaking weight per square inch was 17,136 lbs. (equal to 7.65 tons); in the second 5869 lbs. (equal to 2.62 tons), being very little more than one third of the former (3 × 2.62 = 7.86, being only 0.21 in excess of 7.65).

17. Mr. Tredgold computed that, if the elasticity remained perfect, the strength in these two cases would be as 4 to 1. It will be shown further on, that the ratio of 3 to 1 coincides with the correct theory.

This result, as to the ultimate strength of cast iron to resist tensile forces, was opposed to the conclusions arrived at by the best writers before Mr. Eaton Hodgkinson's direct experiments; but they had *calculated* the tensile strength from the indirect method of experiments on the transverse strength of small bars or beams supported at each extremity. Mr. Tredgold thus made it at least 20 tons per square inch; and Mr. Barlow deduced the value to be upwards of 10 tons.

18. **Malleable Iron.**—*The ultimate resistance of Wrought Iron to a tensile force is about* 20 *tons per square inch for boiler plate, and about* 25 *tons per square inch for the highest quality of rods and bars. Drawn wire, under* $\frac{1}{10}$ *inch diameter, about* 36 *tons per square inch.*

EXPERIMENTAL PROOFS.

19. These have been arranged according to the date at which they were carried on: thus, Mr. Telford and Captain Brown from 1814 to 1817; Mr. Fairbairn, 1837; Mr. Edwin Clarke, 1848; Mr. Hodgkinson, 1848.

Mr. Telford having designed a suspension bridge of a central opening of 1000 feet, for the passage of the river Mersey at Runcorn, determined the detailed dimensions of the several parts from experiments quoted below, from Mr.

Provis's work on the Menai Bridge, to the construction of which the knowledge thus obtained was made subservient; the former having never been carried out.

	lbs.
An iron wire $\frac{1}{10}$th inch in diameter, 100 feet of which weighed 3 lbs. 3½ oz., broke on an average of the first set of 9 experiments with	630
A similar wire on an average of 10 other experiments broke with	634

The extremes of the first set of experiments were 616 and 651 lbs.

	Tons.	cwt.	qrs.	lbs.
1. A bar of South Wales iron, 1⅜ inches diameter, made by S. Homfray, broke with	43	11	0	0
2. do. do., 1½ inch diameter, made by do.	52	15	1	10
3. A bar of Staffordshire Iron, ¾ inch square, broke with	15	5	3	4
4. do. do., 1½ inch, do.	32	6	0	4
5. A bar of Welsh iron, 1 inch square, .	29	0	0	0
6. A bar of Swedish iron, 1 inch square, broke at a flaw, with	29	0	0	0
7. A bar of faggoted scrap iron, made by Mr. Howard of Rotherhithe, 1 inch square, broke with	29	0	0	0
8. A bar of common Staffordshire iron, 1 inch square, broke with	31	0	0	0
9. A bar of common iron, 2 inches in diameter, was torn asunder with . . .	100	0	0	0

The iron wire was torn asunder by a weight suspended from it. The experiments upon the iron bars were made by one of Bramah's presses, at Brunton and Co.'s Chain Cable Manufactory in London.

20. Reducing these for readier comparison to a uniform section of 1 inch square, we find the relative strain which each bore as follows:—

	Tons.	cwt.	
1, reduced to 1 inch square, gives	29	6	Welsh.
2, ,, ,,	29	16	,,
3, ,, ,,	27	3	Staffordshire.
4, ,, ,,	27	10	,,
5, ,, ,,	29	0	Welsh.
6, ,, ,,	29	0	Swedish.
7, ,, ,,	29	0	Faggoted.
8, ,, ,,	31	0	Staffordshire.
9, ,, ,,	31	16	
Mean strength of an inch sq. bar,	29	5⅔	

Mr. Telford, however, considered it safest to adopt the least strength indicated by any bar during those experiments, instead of taking the average, and consequently concluded that a bar 1 inch square would bear up to 27 tons.

21. Captain S. Brown, the inventor both of Chain cables and of the Bar link suspension bridge, has also experimented upon the resistance of wrought iron at his works, Millwall, Poplar. The apparatus used seems to have been exactly similar to that described by Mr. Provis in his work on the Menai Bridge, and probably underrates the strain acting on the specimen under trial.

22. Captain S. Brown's *Experiments on the Tensile Strength of Wrought Iron.*

No. of Experiment.	Description of Iron and Dimensions.	Reduced size at Fracture.	Total weight applied. Tons.	Total weight applied. cwt.	Resistance per sq. inch per ton.
I.	Swedish Iron, $1\frac{5}{16}$ inch sq.	$1\frac{1}{15}$	40	19	23.77
II.	,, ,,	,,	39	15	23.19
III.	,, $1\frac{3}{16}$,,	$\frac{7}{8}$	33	10	23.75
IV.	Russia, $1\frac{5}{16}$ diameter.	1	36	2	26.55
V.	Welsh, No. 3, $1\frac{1}{4}$ square.	$1\frac{1}{16}$	38	1	24.35
VI.	,, common, $1\frac{1}{8}$ square.	$1\frac{1}{16}$	31	0	24.90
VII.	,, No. 3, 2 diameter.	$1\frac{5}{8}$	82	15	26.33
VIII.	,, $1\frac{4}{9}$ diameter.	1.30	43	10	26.34

The mean being 24.9 tons.

The detailed description of these experiments is given by Mr. Barlow; the mean of the four first experiments on the foreign irons was 24.31 tons; of the four last, on the Welsh iron, 25.48 tons per square inch.

23. Mr. Brunel has made experiments on bar iron reduced in section by the operation of hammering.

	Tons.
I. 10 Experiments on iron denominated *best*, and reduced $\frac{3}{8}$th inch square in the middle, gave a mean resistance of	30.4
II. 10 Experiments on iron denominated *best best*, and reduced to $\frac{3}{8}$th inch square in the middle, gave a mean of	32.3
III. 6 Experiments on *best* iron, reduced to $\frac{1}{2}$ inch square in the middle, gave	30.6

In the series No. I. of the experiments given above, one of the specimens was obviously defective, breaking with 25 tons per square inch; it is, however, very properly included in deducing the mean resistance; had it been omitted, the average would have been increased from 30.4 to 30.98.

These results, so high when compared with those of the preceding Tables, may, perhaps, be explained by the fact of the reduction in section by hammering, and would then be analogous to the increase of strength observable in drawn iron wire.

The results of Mr. Telford's experiments on iron wire are found in the Table appended.

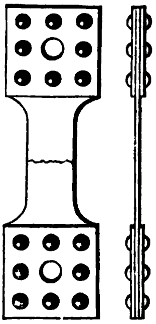
24. Mr. Telford's *Experiments on the Tensile Strength of Iron Wire.*

	Diameter of wire in inches.	Area in square inches.	Breaking weight in lbs.	Breaking weight in tons per sq. inch.
I.	$\frac{6}{70}$ = 0.0857	0.0058	531	35.7
II.	$\frac{1}{5}$ = 0.2	0.0314	738	42.0
III.	$\frac{1}{10}$ = 0.1	0.007854	277	42.9
IV.	$\frac{6}{100}$ = 0.06	0.00283	157	38.1
V.	$\frac{1}{21}$ = 0.0476	0.0018	630	35.8
VI.	$\frac{1}{10}$ = 0.1	0.007854	634	36.1
	Mean, ...			38.4

The irregularities being very great, it is prudent to take the strength at 36 tons, as in § 18.

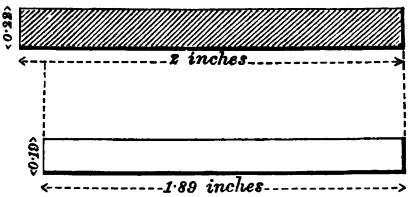
This material, almost exclusively used for suspension bridges on the Continent, and in the patent iron wire rope, has found in this country a very useful and extended application, in the standing rigging of vessels; in lifting material in the shafts of mines; and on railway inclined planes worked by stationary power.

25. Mr. Fairbairn ("Useful Information for Engineers") has also experimented on the strength of wrought iron plates upon a scale of such magnitude as to supply sound practical data, and lead to an economical as well as very extended use of the material. The apparatus used was very nearly the same as that described §§ 4, 5.



In these experiments all the plates were of uniform thickness, and of the form shown in the preceding woodcut.

The ends had plates riveted to them on both sides, to render them inflexible; they had holes bored through them perpendicular to the plate, in order to connect it by bolts with the apparatus for tearing it asunder, which was designed to occur at the centre, in the waved line, where it was made narrower than the rest. The centres of the holes were in a direct vertical line through the middle of the piece.



The upper woodcut is added to show the actual dimensions of the transverse section of a specimen under trial; and the lower the reduced dimension before yielding to the breaking weight, as noted in No. I. in the Table below with the asterisk.

26. *Low Moor Yorkshire Plate Iron.*

No. of experiments.	Description of plate, and dimensions in the middle.	Weight laid on in lbs.	Reduced dimensions in middle of plate, through weight laid on.	Breaking weight in lbs.	Mean breaking weight.
	Drawn in the direction of the fibre.				
I.	Area, 2.00 × 0.22 = 0.44	24043	1.96 × 0.21		
	square inch, . .	25531	1.89 × 0.19*	25531	25400 lbs., or 25.77 tons per square inch.
II.	. . . .	23571	Reduced.		
		24747	1.93 × 0.18	24747	
III.	. . . .	25923	1.94 × 0.18	25923	
	Drawn across the fibre.	23179	Altered.		
IV.	Area, 2.00 × 0.22 = 0.44	24355	1.99 × 0.215		
	square inches, . .	25923	. . . . .		27099 lbs., or 27.49 tons per square inch.
		27099	2.2 × 0.19	27099	
V.	Same area, ...	24355	Stretching.		
		26315	2.15 × 0.20	26.315	25662 lbs., or 26.037 tons per sq. inch.
VI.	. . . .	23571	2.25 × 0.20	23571	
	Thicker plates drawn in direction of fibre.				
VII.	2.00 × 0.26 = 0.52 in.	27099	Thickness 0.25	27099	26511 lbs., or 22.76 tons per square inch.
		25923	1.96 × 0.24	25923	

In Nos. I. II. III. the plates were laminated, as if formed of three or more plates, the external ones being thinner than the internal.

Nos. IV. V. VI. did not break at the narrowest part; a circumstance the more anomalous, as there did not appear to be anything in the apparatus to cause it.

No. VII. was very uniform in texture. The fracture showed a great want of regularity; about one-third of the area had the appearance of steel. All the other plates appeared to be uniform, but laminated, as above mentioned.

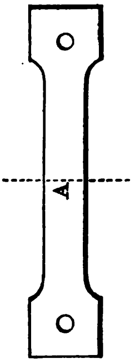
27. Other Tables, equally detailed, are given by Mr. Fairbairn, on plates from the Iron Works in Derbyshire, Shropshire, and Staffordshire, of which he gives the following summary:—

	Mean breaking weight in the direction of the fibre, in tons per sq. inch.	Mean breaking weight across the fibre, in tons per square inch.
Yorkshire plates.	25.770	27.490
Yorkshire ,,	22.760	26.037
Derbyshire ,,	21.680	18.650
Shropshire ,,	22.826	22.000
Staffordshire ,,	19.563	21.010
Mean, ...	22.519	23.037

28. Mr. Edwin Clarke gives the following Table of experiments made at the Britannia Bridge, on the rolled plates called boiler plate, and on the rivet iron employed in that construction. The mean being 19.6 tons.

	1	2	3
	Description and Dimensions of Boiler Plate.	Breaking weight per square inch.	Ultimate Extension in parts of the length.
		Tons.	
I.	Plate $\frac{11}{16}$th inch thick, neck $1\frac{1}{2}$ inch long, selected as bad iron; fracture bright and crystalline, brittle; broke readily with a blow from a hammer.	22	
II.	Same plate.	21	$\frac{1}{40}$
III.	Plate $\frac{1}{2}$ inch thick, neck 6 inches long, selected as bad iron; containing two laminæ of crystalline metal, $\frac{1}{3}$rd of the whole section.	18	$\frac{1}{180}$
IV.	Plate $\frac{1}{2}$ inch thick, neck 5 inches long, selected as a good plate; about $\frac{1}{10}$th of section crystalline.	19	$\frac{1}{48}$
V.	Plate $\frac{1}{2}$ inch thick, neck 4 inches long, iron perfectly uniform and fibrous; supported the weight 15 minutes.	21	$\frac{1}{8}$
VI.	Plate $\frac{11}{16}$ inch thick, neck 5 inches long, iron good; $\frac{1}{40}$th of the section crystalline.	19	$\frac{1}{36}$
VII.	Plate $\frac{1}{2}$ inch, neck 5 inches; iron fibrous, except $\frac{1}{50}$th of section.	18	$\frac{1}{23}$
VIII.	Plate $\frac{1}{2}$ inch, neck 50 inches.	19.6	$\frac{1}{57}$
IX.	Plate $\frac{5}{8}$ inch, neck 50 inches.	19.3	$\frac{1}{33}$
X.	Plate $\frac{1}{2}$ inch, neck 7 inches.	19.6	$\frac{1}{19}$
XI.	Plate $\frac{1}{2}$ inch, neck 7 inches.	20.2	$\frac{1}{24}$
XII.	Plate $\frac{1}{2}$ inch, neck 50 inches.	18.7	$\frac{1}{59}$

29. The specimens were carefully reduced to a uniform neck, in the following form, the sectional area of the neck being always 1 square inch.



Section at A, 1 square inch.

The bars were suspended from a shackle and broken without the intervention of a lever, by direct weight placed in a scale.

"The examination of this Table will show a very great constancy in the ultimate resistance. The ultimate extension, on the contrary, is extremely irregular; indeed, some of the brittle crystalline irons selected as bad, and which fractured suddenly without much increase of length, actually supported a greater weight than the more fibrous and ductile iron."

30. It may be instructive now to compare the results obtained by the different experimenters, which have been quoted above, upon the ultimate tensile strength of wrought iron.

Mr. Telford,	1814 to 1817	§ 20 gives	27	tons per sq. in.	Bars.
Capt. S. Brown,		§ 22 ,,	24.9	,, ,,	
Mr. Fairbairn,	1837	§ 27 ,,	23	,, ,,	Plates.
Mr. E. Clarke,	1848	§ 28 ,,	19.6	,, ,,	

This gradual reduction is very remarkable, but has not been commented upon by any author; it may be mentioned, however, that the first two results were obtained by the hydraulic press, or by the action of compound levers, after the manner of the weigh bridge, both which have a tendency to overrate the results. The third by Mr. Fairbairn, by means of a lever of 8 or 10 to 1. The last only, those by Mr. Edwin Clarke, being obtained by a weight in a scale suspended directly from the specimen itself, in which case it is only necessary to take the precaution that the line of direction of the applied force pass through the centre of figure of the piece under trial. No deterioration in the quality of iron to such an extent can be supposed to have taken place, but the earlier specimens were probably selected with the utmost care, and of the very highest quality. The numbers adopted in the Proposition, § 18, are from Mr. E. Clarke, and, being the lowest, are at least the safest, and very trustworthy.

31. Malleable iron is, in the process of rolling, given a fibrous structure, the direction of the fibres being the same as that in which the metal is rolled. Experimenters always apply the testing forces in this direction, and in the Table given above

the line of action of the force was coincident with that of the fibre. In order to prove whether any difference of resistance existed in this direction, compared with that at right angles to it, that is, across the fibres, the following experiments were undertaken by Mr. E. Clarke.

	Ultimate strength when drawn in the direction of the fibre, in tons per square inch.	Ultimate strength when drawn across the fibre, in tons per square inch.
I.	19.66	16.93
II.	20.20	16.70
	Average, 19.93	16.81

The result is 18 per cent. in favour of the power of resistance in the direction of the fibres, that is, in the direction in which the iron has been rolled; but Mr. Fairbairn, in his experiments, with the same object, arrived at a very different result, namely, that the power of resistance of the plate was very nearly the same, whether the tensile force acted in the direction of the fibres, or across them, § 27. This question, as well as that of the relative strength of Plates, Bars, and Wire, is evidently connected with the original selection of the material, and subsequent careful manufacture, rather than with any physical difference in the iron itself.

32. Mr. E. Hodgkinson experimented upon a rod of annealed iron, the quality being that called in trade *best*, having a diameter of 0.517 inch (a mean of 14 measurements), giving an area of 0.2099 square inch, the length being 49 feet 2 inches. This experiment, which will be quoted in full further on, under the head of Elasticity, gave an ultimate resistance of 23.817 tons per square inch.

An experiment made by Mr. Mare, at his works at Blackwall, upon a round rod, of $\frac{7}{8}$th inch diameter, and 5 feet long, made of scrap rivet iron, and of unusually good quality, gave an average resistance of 24 tons per square inch. These are the highest results of recent experimenters.

33. We may now perceive, by a comparison of the ultimate resisting powers of Cast and Wrought Iron to tensile forces, why the latter ought always to be adopted in practice. The same transverse area, as, suppose, one square inch in each,

having very nearly three times the strength when of Wrought Iron to that which it has when consisting of Cast. Iron, the numbers being about 20 tons for the former, and 7½ for the latter. But Wrought Iron has also a very great superiority in possessing a high degree of ductility; being thus secured from that suddenness of fracture characteristic of Cast Iron, which, in resisting direct tension, breaks without warning us by any gradual small change of form. And as to the relative cost, we may take wrought-iron forgings as about double the price of large castings not requiring any very complicated patterns. It will appear, moreover, that the specific gravity of these descriptions of iron is very nearly equal.

PRACTICAL APPLICATIONS AND EXAMPLES—WROUGHT IRON.

34. In applying to the actual practice of construction the knowledge of the ultimate resisting powers of any material, as obtained by experiment, the engineer must, in all his designs, keep in view the great objects of permanency, stability, and safety, and therefore assign to every part such dimensions and form that it shall be acting far within its ultimate power. But the question at once arises, How far within?—what fraction of the ultimate must be the actual resisting force called forth, and this, not only in the case of tensile, but of all the other forms of resistance occurring in structures mentioned in Sections 1 and 2.

For this no absolute rules can be set down; a careful judgment and a sagacious use of existing precedents must alone guide us here: a knowledge not only of existing successful works—which, from being, as may often occur, stronger than strong enough, have perhaps purchased their stability at the expense of economy, and are consequently, so far, not to be followed,—but also from the records of past failures, which are in truth but unintended and expensive experiments. Thus it would appear, that there is yet a knowledge required, above and beyond that given by experiment, which it cannot teach, and that is the proper application to practice of the facts established by that means.

As preliminary to designing successfully, it would, therefore, appear necessary—

First—To calculate from mechanical principles the strains brought upon every part by the weight of the fabric itself, and by its particular position in it; and also the additional strain caused by any load the structure may have to sustain, whether stationary or passing.

Secondly—To obtain from experiment a safe average value for the ultimate resistance of the material; and—

Thirdly—From good precedents, by judgment, to adopt what may be named the coefficient of stability, or that fraction of the ultimate resistance beyond which it would be injudicious to strain the material.

35. As to the coefficient of stability, so far as the tensile resistance of wrought iron—the material we are at present considering—is concerned, we learn from the largest existing works of a permanent character, that no part in tension is strained so as to call forth a resistance greater than about five tons per square inch of transverse section, that is, ¼th of the ultimate resistance.

The main chains of the Menai suspension bridge are resisting 5.4 tons per square inch; the bottom of the Conway tubes, 4.5; and of the Britannia tubes, 5.62 tons per square inch. In the temporary work of the lifting of the latter, the bar-link chains, hung from the cross heads of the hydraulic presses, had to resist 8.3 tons per square inch (Clarke on "Brit. Bridge," pp. 612, 760, 761); but it would have been very difficult, from the confined area in which that operation had to be carried on, to have found the requisite space for them, if enlarged: and the temporary nature of the duty they had to perform may have permitted this great strain being used. The Boyne viaduct was so designed that 5 tons per square inch was the maximum strain from the greatest load, which was assumed to be 2 tons per foot of span, plus the weight of the structure itself, and this after having deducted the area of the rivet-holes from the section of the part resisting tension.

36. Each different material in use in the constructive arts has peculiar modes of combination and application answering to its natural properties. It is thus of timber, in the use of which particular methods of junction and fastening are employed, and so also of the two different varieties of iron,—cast iron and wrought iron. The former is obtained of the required dimensions and form by using its property of becoming *fluid* at a high temperature, and thus, being run in a molten state into moulds duly prepared from patterns; and very frequently, when the dimensions of the piece are greater than may be

convenient or practicable for a single casting, numerous pieces are joined by screw-bolts passing through flanges, the surfaces of which are fitted by means of truly planed fitting strips.

Wrought iron is, on the other hand, given the requisite shapes by its properties of being welded together at a white heat; of being rolled at the rolling mill, and of being united by riveting. By the first we obtain from the steam-hammer—anchors, shafts, and crank-arms, wrought-iron ordnance, &c. By rolling is produced the railway bar iron, angle iron, boiler plates, plates for bridges, &c., and generally such forms as have a uniform transverse section in a plane perpendicular to the axis of the bar or plate. By riveting we are enabled to join the rolled plates so as to construct iron ships, steam-engine boilers, gas-holders, lattice and tubular bridges, all which are of such size and shape as could not be obtained otherwise than by the due adjustment and firm union of numerous relatively small separate pieces, the largest dimension of which is limited by the existing state of manufacturing power at iron-works.

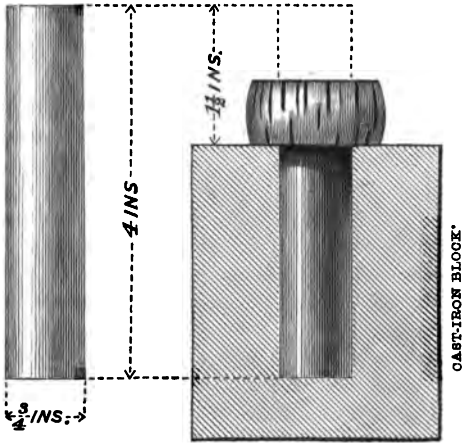
Having then determined the ultimate resisting power of wrought iron to tensile forces, we must naturally proceed now to determine its ultimate resistance in combination—i. e. when many different pieces have been united together—as compared with that of the plates or bars themselves.

In the main chains of bar-link suspension bridges, wrought iron may also be joined by transverse bolts through eyes at each extremity of the links, rigidity at the joints not being essential. This method of junction is explained and illustrated in § 59.

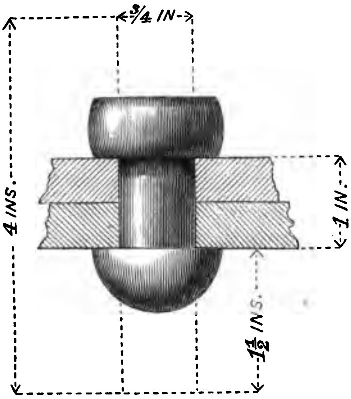
37. It is but recently, since the rapid extension of the Railway system, that Civil Engineering has become so much indebted, in its great continual progress, to the introduction amongst its constructive resources of wrought-iron plates united by riveting, by which tubes, girders, and beams, &c., have been constructed. This process of uniting plates had hitherto been a peculiar feature of boiler-work and iron ship-building.

38. In riveting, the pieces of plate iron are overlapped, Fig. 6; and holes having been punched or drilled in each plate separately from a template, so as exactly to correspond, they are placed over each other, and temporarily held together by cotter pins; the rivets (the manufacture of which must now be described) are dropped red hot into the holes, and closed upon the plates by hammering.

The rivets are thus made:—A bar of iron, which may in ordinary cases vary from about $\frac{7}{8}$ths of an inch to $\frac{1}{16}$th in diameter, according to the plates it is intended to unite, and which, at an average, we will suppose to be $\frac{3}{4}$ths of an inch in diameter, is cut up in a cold state into lengths by a pair of shears: for joining two $\frac{1}{2}$-inch plates, this length is about 4 inches. These short lengths are then made red hot on the floor of a kind of reverberatory furnace, and in this state dropped into a block of cast iron, having holes drilled in it about $1\frac{1}{2}$ inches shorter than the length of these pieces, which, therefore, project $1\frac{1}{2}$ inches above the surface of the cast-iron block. The woodcut represents at half size the short piece of round bar; and also the same placed in the block or mould after a few blows from a heavy hammer have flattened out the projecting part into a head. The block is supposed to be cut by a vertical plane through the centre of one of the holes; the dotted lines show the projection of the original length. "The head frequently presents the cracked appearance round the edge, as shown in the engraving, the longitudinal fibres slightly opening; this is not detrimental to the strength of the rivet, and is regarded as characteristic of a tough and fibrous iron." The head is now most generally produced by machinery; instead of the blows of the hammer, a steady pressure is produced by a flat disc, attached to a slide, moving vertically by means of a cam or excentric; many heated rods are dropped into one mould with several holes, and passed in rapid succession beneath the disc, which, descending to within a *constant* distance



of the mould, flattens out the heads to a uniform thickness; the heads sometimes are given a conical form by the use of a hollow instead of a flat disc, insuring also a uniformity of size. A man and boy can by this machine form from 300 to 450 rivets per hour. This woodcut represents a section of the



overlapping ½-inch plates, and the closed rivet, which, having been reheated in a portable forge or reverberatory furnace, is placed in the hole, and closed by the riveters, the dotted lines above and below showing the original round bolt of the rivet projecting 1½ inches. A gang of men for riveting consists of a holder-up, and two riveters, with boys to supply them with rivets from the furnace. The holder-up presses against the head, generally by a heavy cast-iron hammer, and the two riveters with their hammers commence with a few blows on the plates around the rivet, to bring them into closer contact; then, striking alternately on the red hot rivet, they drive as much as possible into the hole through the plates, so as to completely fill it (called *upsetting*), after which a few blows flatten out the head, which is finally given a hemispherical form by a cup-shaped tool called a *snap*, held on the rivet and struck by the riveter, like the ordinary swaging. The plates are thus forcibly held together, and the rivet, on cooling, contracts and draws the surfaces of the plates into the closest contact.

The plates, when laid upon each other preparatory to riveting, however well they may have been rolled in the manufacture, are not so truly plane surfaces as if they had been fitted and prepared in the planing-engine; the ductility of the wrought iron enabling it to adapt itself to these small inequalities of surface, and also to bear the blows of the riveting hammers: we may thus understand how it is that cast iron cannot be joined by the method of riveting, for even if the pieces of metal had been planed and fitted, the hammering would endanger their being cracked.

39. If we suppose a tensile force applied to tear asunder two plates thus riveted together, Fig. 6, it is evident that, either the plate must be torn asunder in its weakest transverse section A, B, or else the rivets will be cut across in the common plane of the two plates by a shearing action of the tensile force, as shown in Fig. 7. Now, in the *best* arrangement of the relative dimensions of plate and rivets, we must have each on the point of yielding simultaneously, for if the rivets only be sheared across, it is evident that the transverse section of the plates was the stronger, and that some superfluous iron might have been removed from them without lowering the real power of resistance in the joint, which in this case is measured by the resisting power of the rivets,—thus saving material; or we might have increased the diameter or the number of the rivets, up to an equality of resisting power with that of the plates, thus augmenting the ultimate strength of the joint. In like manner, if we had supposed the plate to yield, and not the rivets, then, as before, without diminishing the strength of the joint, which is now measured by that of the plate, we might reduce the rivets, or we might have increased the transverse area of the plate, in both cases raising the ultimate strength of the joint.

40. To design a perfect joint, we must, therefore, in addition to our knowledge of the tensile resistance of the plates, experimentally determine the power of rivet iron to resist a shearing force; whence, being given the number of rivets in a joint of a certain width and thickness of plates, we can calculate the diameters, so that their total resistance to shearing may be equal to the tensile resistance of the transverse section of the plates diminished by the rivet holes.

41. Mr. Edwin Clarke has, from experiments on the shearing strength of bars and rivet iron, deduced the following simple laws:—

I. The ultimate resistance of any bar to a shearing force is proportional to the sectional area of the bar thus severed.

II. The ultimate resistance of any bar to a shearing force is nearly the same as the ultimate resistance of the same bar to a direct longitudinal tensile strain.

The machine used is represented in Fig. 8; it consists of a wrought-iron lever ¾ths of an inch thick, and 7 feet long, working round the fulcrum *a*, in a block of cast iron, slotted as shown in the transverse section, Fig. 9, which is taken through the dotted line in Fig. 8. The fulcrum is formed by a strong pin passing through both the lever and cast-iron block; at a distance of 6 inches above the fulcrum a hole was drilled through the cheeks of the block, and a corresponding hole in the lever, when in its horizontal position, enabled the experimenter to insert the bar to be tried, either through and through, or else only through one cheek and the breadth of the lever. In the former case, when the weights were laid on at the opposite end of the lever, the bar was cut across simultaneously in two parallel sections, a piece ¾ths of an inch in length being cut out of the middle of the bar in each experiment, in the latter case in one place only; a piece ¾ths of an inch in length being cut off the end of the bar at each experiment. It is evident that in this apparatus we have a bell-crank or bent lever; and as the distance from the point of application of the weights to the fulcrum was 6 feet, that the ratio of the arms was 12 to 1.

The experiments were made on rods of rivet iron, ⅞ths of an inch diameter; area, 0.6013 square inches, of excellent quality, and perfectly fibrous.

Experiments by E. CLARKE, *on the Single Shearing of Bars of Rivet Iron; Diameter, ⅞ths of an inch; Area,* 0.6013 *sq. inch.*

Experiment.		Weight in tons per square inch of Section.
1	Weight required to shear the bar = 2956 lbs.	26.1
2	" " "	23.9
3	Four bars, each loaded with the same weight, but the additions to the weight were too sudden, ...	21.1
4	Six bars of iron of a different quality, . .	25.9
	Mean, ...	24.15

Experiments on the Double Shearing of Bars of Rivet Iron, as above; Diameter, ⅞ths of an inch.

Experiment.		Weight in tons per square inch of Section.
5	Area of the two sections, 1.2026 square inch,	22.9
6	” ” ”	21.6
7	” ” ”	21.6
8	Diameter, 27/32ths of an inch, ”	22.5
9	” ” ”	22.5
10	” ” ”	22.5
11	Bar, diameter, ⅞ths of an inch, ”	21.6
12	” ” ”	21.6
	Mean, ...	22.1

The general mean gives 23.13 tons per square inch, as necessary to shear across a single rod of rivet iron of good quality.

The ultimate tensile strength of these same bars was also found to be 24 tons; hence, their resistance to single shearing was nearly the same as their ultimate resistance to a tensile strain.

To avoid any anomalies from the use of the lever, or from the fitting of the pin loosely in the hole, two plates ⅝ths of an inch thick were now riveted together by a single rivet, ⅞ths of an inch diameter, and the rivet was sheared by suspending actual weights from the plates; the rivet then sustained 12.267 tons, or 20.4 tons per square inch.

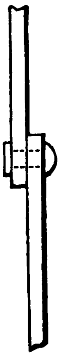
Three plates were then united by a similar rivet, and the rivet was sheared in two places by the centre plate. The ultimate weight suspended from the rivet was 26.8 tons, or 22.3 tons per square inch of section.

The woodcut on opposite page, No. 1, represents in section plates riveted for a single shear; No. 2, for a double shear; No. 3, for a treble; No. 4, for a shearing at four places.

42. These experiments supply all the information that was needed to enable us to construct a joint of the best relative proportion of plate to rivet. For (supposing the quality of iron to be the same in each) it is evident that the area of the

weakest section of the plates, namely, that through the centres of the outermost row of rivets, must be equal to the sum of the transverse areas of all those same rivets, for thus the resistance to tensile forces of the sum of all the rivets will be equal to that of the plate diminished by the rivet-holes; and, the ultimate tensile and shearing resistance of the rivets being equal, the latter is evidently equal to the ultimate tensile resistance of the plate, less the holes punched for the rivets.

No. 1.



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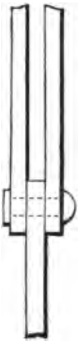

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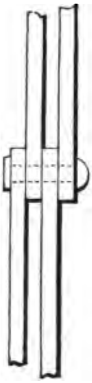

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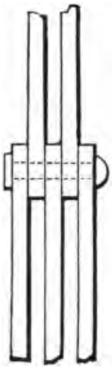
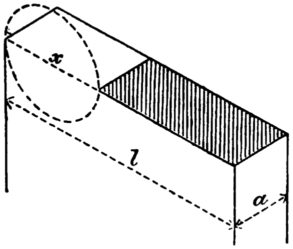




43. As the distance from centre to centre is the same in all the rivets in any piece of work, and the edge of the plate from the centres of the extreme rivets half that distance, we may simplify the consideration by taking the case of one rivet only, in a portion l of the breadth of the plate equal to the distance from centres, as in the woodcut. The thickness of the plate a being also given, we have only to calculate the diameter of a circle x such that the area of it may be equal to the transverse section of the plate, or $a \times (l - x)$, from whence we have

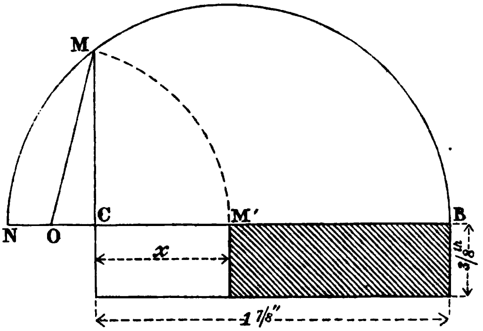
$$\frac{11}{14} x^2 = al - ax,$$

and, dividing by $\frac{11}{14}$,

$$x^2 = \frac{14a}{11}\,l - \frac{14a}{11}\,x;$$

putting $k = \frac{14a}{11}$, and transposing, we have, $x\,(x + k) = k \times l$.

Hence we may solve this geometrically, for it is equivalent to producing a given line k, so that the rectangle under the produced part x, and the whole line $k + x$, shall be equal to a given quantity, namely, $k \times l$. The diameter of the rivet thus depends upon the thickness of the plate, as well as upon the assigned distance l, between the centres of the rivets.



In the woodcut a portion of the plate is represented in section, through the line of centres of rivets, at the full size, the thickness being supposed $\frac{3}{8}$ths of an inch, and the distance CB from centre to centre of rivets $1\frac{7}{8}$ inch, as figured; the solid portion of the plate having diagonal hatchings, and being equal in area to a circle described on x as diameter. The value of k with the above dimensions is 0.4773 inch, or $\frac{14}{11} \times \frac{3}{8}$ths of an inch, also $k \times l = 0.4773 \times 1.875 = 0.8947$ square inch. We have now to find a length x such that, being added to 0.4773 inch, and multiplied into the sum, the product shall be equal to 0.8947 square inch.

In BC produced, lay off CN equal to k, on NB describe a semicircle, and at C erect the perpendicular CM to cut the circle at M; then $k \times l = \overline{NC \times CB} = (CM)^2 = 0.8947$ square inch. If then we find a point M′ such that $\overline{NM' \times CM'} = (CM)^2$, we determine the diameter of rivet CM′. Bisect NC in O, and join OM; with O as centre, and OM as radius, describe an arc cutting CB in M′, then M′ is the point required. For the equal squares $(OM)^2$ and $(OM')^2$ are respectively equal to $(OC)^2 + (CM)^2$ and $(OC)^2 + \overline{2\,OC \times CM'} + (CM')^2$. Take from each the equal quantity $(OC)^2$, and as $NC = 2\,OC$, we have,

$$(CM)^2 = \overline{NC \times CM'} + (CM')^2 = \overline{NM' \times CM'},$$

satisfying the above equation, and giving $CM' = 0.735$ inches, nearly ¾ths of an inch, identical with the sixth example, § 44.

The diameter of the rivet is generally about $\frac{1}{16}$th of an inch less than that of the hole drilled or punched for it; this space is, however, completely filled in good riveting from the *upsetting* of the round bar by the hammering in closing.

44. The following instances of the dimensions of riveted work will show what has been adopted in practice.

The wrought-iron gates of the Dublin Graving Docks (Mallet's Patent) have rivets ¾ths of an inch diameter, 2¼ inches centre to centre; this gives 1½ inch length of plate between contiguous rivet-holes, or double the diameter of the rivet, a proportion that may very frequently be observed. The thickness of the plates was about ⅝ths of an inch.

In locomotive boilers we have the following examples:—Diameter of boiler outside, 3 feet 6 inches; plates, $\frac{5}{16}$ths of an inch thick, overlapping; rivets ⅞th of an inch diameter, and 1¾ths of an inch apart. ("Description of Stephenson's Patent Locomotive," p. 7; Weale's Tredgold "On the Steam Engine.")

Diameter of boiler, 3 feet 6¾ inches; plates, ⅜ths of an inch thick, overlapping; rivets ¾ths of an inch in diameter, and 2 inches apart, centre to centre. ("Specification of Passenger Engines, Caledonian Railway," *ibid.*)

Diameter of boiler, 4 feet high, 3 feet 10 inches wide; plates, $\frac{7}{16}$ths of an inch thick, overlapping 2¼ inches; rivets ¾ths of an inch in diameter; and 1¾ths of an inch apart centres. ("Description of Hawthorn's Express Engine," *ibid.*)

Boiler, 3 feet 6 inches in diameter inside; plates, ⅜ths of an inch thick; jump-jointed with covers or welts; rivets, ¾ths of an inch in diameter. ("Description of Locomotive Engine by Sharp, Roberts, and Co.," *ibid.*)

At Inchicore Locomotive Workshops, when diameter of boiler does not exceed 3 feet 10 inches, the plates are ⅜ths of of an inch thick; the rivets ¾ths of an inch in diameter, and 1⅞ths of an inch centres.

The boiler plates are also chipped perfectly true by the cold chisel, and caulked both inside and outside—i. e. a portion of the plate is driven into the joint between the welt and the but-jointed plates; this renders them perfectly steam-tight.

The bottom plates of the Great Britain steam-ship are ¾ths of an inch thick; the rivets are 1 inch in diameter, and inserted at distances of 3 inches centre.

Such are the proportions and forms of a few particular cases, in which it is highly important that the several parts in the construction should be united with the utmost strength. The following Table, deduced from experiments and actual practice by Mr. Fairbairn ("Useful Information for Engineers," Appendix, p. xxix.), gives the requisite dimensions of riveted work, in terms of the thickness of the plates, extending from $\frac{3}{16}$ths to $\frac{12}{16}$ths of an inch, and thus embracing every case that usually occurs in practice.

TABLE *exhibiting the Strongest Forms and Best Proportions of Riveted Joints.*

Thickness of plates in inches.	Diameter of rivets in inches.		Length of rivets from the head in inches.		Distance of rivets from centre to centre in inches.		Quantity of lap in single joints in inches.		Quantity of lap in double-riveted joints in inches.
0.19 = $\frac{3}{16}$	0.38	2	0.88	4.5	1.25	6	1.25	6	For the double-riveted joint, add two-thirds of the depth of the single lap.
0.25 = $\frac{4}{16}$	0.50		1.13		1.50		1.50		
0.31 = $\frac{5}{16}$	0.63		1.38		1.63	5	1.88		
0.375 = $\frac{6}{16}$	0.75		1.68		1.875		2.00	5.5	
0.50 = $\frac{8}{16}$	0.81	1.5	2.25		2.00	4	2.25	4.5	
0.63 = $\frac{10}{16}$	0.94		2.75		2.50		2.75		
0.75 = $\frac{12}{16}$	1.13		3.25		3.00		3.25		

It will be perceived that, except in the third column, the same multiplier or proportion does not continue; in the rest they decrease as the plates increase in thickness.

Let us take a plate $\frac{3}{8}$ths of an inch thick, and seek from this Table the diameter of rivet and other dimensions, when the plates are joined in the strongest manner. The diameter of the round bolt of which the rivet is formed will be twice the thickness of plates, or $\frac{3}{4}$ths of an inch. The distance apart from centre to centre will be 1 inch and $\frac{7}{8}$ths, or 1.875—i. e. five times the thickness of the plates.

Again, in joining plates $\frac{3}{4}$ths of an inch thick, the diameter of rivet should be once and a half that thickness, or 1.13 inch, and the distance apart centres, four times the diameter, or 3 inches; and so of the other parts.

45. About 30 per cent. of the section of a plate, as it is before having been drilled or punched for the rivets, may be considered, approximately, as removed, and to that extent it would be weakened for all purposes of tensile resistance, were it

not for the other forms of joint, which have been devised in lieu of the simple overlap, with a single line of rivets.

These are:—The double riveted joint, with the rivets placed zig-zag. The chain-riveted joint. The joint secured by V-formed overlaps or cover plates, with rivets diminished in number in each successive parallel line.

The joint in which it is proposed to employ oval instead of circular rivets. The joint, with plates thickened at the ends, where pierced for rivets. In all these it may be remarked that any comparative increase of strength is obtained by increased weight of material, and all in different degrees fall short of that of the plate.

The double, treble, &c., riveted joints, overlapping, are shown in Plate II., Figs. 9*b*, &c. The relative tensile resistance of the unpierced plates, the same when joined by double riveting and single riveting, are as the numbers 100, 70, and 56,

Chain-riveted butt joints of various forms are shown in Figs. 9*g*, &c.

The joint with plates having a thickened edge is shown in Fig. 9*k*, if the rivets are 3 diameters from centre to centre, the thickness of plate to that at the thickened edge must be as 2 to 3 to make the joint of the same strength as the plate.

46. The drawings, Plate III., Figs. 9*n*, &c., are from a series of specimens in the Model-room of this School. The plates in each of them are 18 inches wide, and $\frac{3}{8}$ths-inch thick, having therefore a transverse section of 6.75 square inches. The rivets being $\frac{5}{8}$th-inch diameter have a transverse areaof 0·3 square inches resisting shearing, and the loss of plate at each rivet hole is 0.234 square inches, Fig. 9*w*.

In No. I. the plates are overlapped, and V-formed at the ends, with 22 rivets through each plate. The drawings that follow to 9*s* show in end view and elevation the effects of a supposed fracture through the succcessive lines of rivets, commencing at *a*; a statement is also given of the sum of the area of the plate and rivets at each of these lines.

The weakest part of the joint is, therefore, through the plate at *a*, where it is less than that of the unpierced plate by one rivet hole, or the $\frac{1}{29}$th part, and to that extent the efficiency of the plate is sacrificed.

In No. II. the plates, butt-jointed, are united by two covers, each $\frac{5}{16}$ths inch thick; the rivets being in double shear, are, therefore, about half as numerous in each plate end—12 instead of 22.

In No. III. the cover plates are, together, so much thicker than the plate, that their sectional area through the line of rivets *e* or *d* is as great as that of the plate through *a*. To compute this we have—x being the sum of the thicknesses of the cover plates—

$$14.875\ (= 18'' - 5 \times 0.625) \times x = 6.516 \text{ square inches,}$$

$$\text{Dividing, } x = \frac{6.516}{14.875} = 0{\cdot}44 \text{ inches,}$$

or about ¼-inch for each cover. The cover plates in

No. II. contain 148.5 cubic inches ;
No. III. „ 103 „

nearly 3 to 2 in favour of No. III.

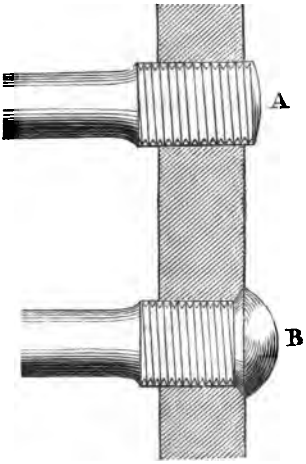
The methods of jointing the plates in the flanges of girders is given in No IV., Fig. 9*v*.

The joint formed with rivets of an oval section, and having the smaller diameter parallel to the edge of the plate, is illustrated in Fig. 9*x*; compared with the circular rivet the distance is increased by one-sixth, and therefore the area of the oval should be greater than that of the circle in the same proportion; the diameter of the circle being ¾-inch, and the smaller diameter of the oval equal to ½-inch, its larger diameter must be 1⅛ (= 1.125) inch to give an equal area for shearing; but as the plates are now one-sixth stronger, we must make the area of the oval rivet stronger in the same proportion, or 1·30 in the larger diameter; the overlap of the plates must evidently be greater than when circular rivets are used, the distance from the edge being maintained the same.

47. Analogous to rivets are the stays used in the fire-box, Fig. 9*y*, of locomotive and marine boilers. The fire-box, consisting of the internal flat plates on which the fire plays—always made of copper, from its ductility and high conducting power—and the outer plates of wrought iron, parallel to these, about 3 inches apart, would certainly yield to the pressure but for the numerous stay-pins inserted transversely, which, being screwed into each plate, and the projecting heads hammered down as in riveting, hold the two parallel flat plates, so that no change of shape can occur from an internal pressure ; each stay has to act

against the pressure upon 16 to 27 square inches of surface, according to the distance apart at which they are placed.

48. Mr. Fairbairn experimented on the resisting powers of the iron screwed into a portion of an inside copper plate of a locomotive engine, which had burst in the fire-box. The stay marked A in the woodcut, $\frac{3}{4}$ths of an inch in diameter, not riveted, required a force of 18260 lbs. = 8.1 tons, to strip the screw and draw it out of the copper plate $\frac{6}{16}$ths of an inch thick; and the stay B, of exactly the same dimensions, but riveted over the end, required a force of 24140 lbs. = 10.7 tons before it was separated, showing an increase of resisting power of about 24 per cent. If such stay had therefore to support the fire-box in which they were placed in squares of 4 inches in the side, the resisting power would be equal to a pressure of 1500 lbs. per square inch; if of the larger size mentioned above, the squares being 27 inches area, the resisting power would be 894 lbs. per square inch.



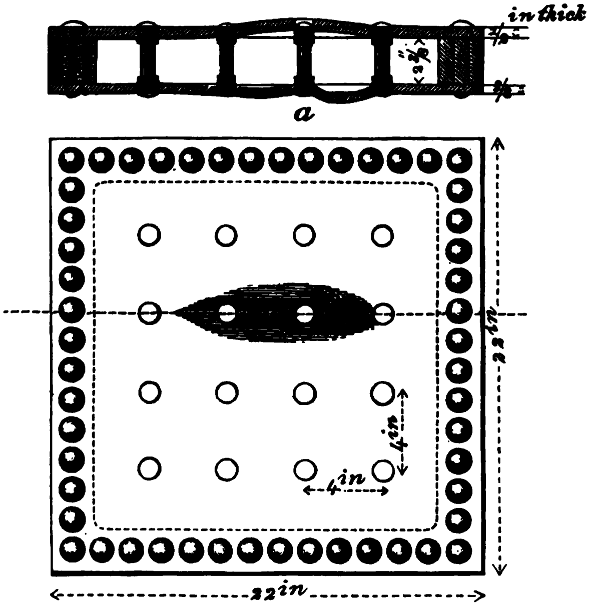
Further experiments were tried on iron stays, $\frac{3}{4}$ths of an inch diameter, screwed and riveted into *iron* plates, and a copper stay $\frac{3}{4}$ths of an inch in diameter (which, as lasting double the time that those of iron are found to do, are made use of in those parts of the fire-box where they are not accessible, and where, consequently, if of iron, they could not be readily renewed), screwed, and riveted into a *copper* plate, both plates being $\frac{3}{8}$ths of an inch thick.

The iron stay, screwed and riveted into iron plate $\frac{3}{8}$ths of an inch in thickness, was torn asunder transversely with 28760 lbs. = 12.5 tons, both screw and plate remaining perfect.

The copper stay, elongating with the successive weights applied, finally broke with that of 16265 lbs. = 7.2 tons: the ultimate elongation being 0.56 inch in a length of 3 inches.

49. In order to render these experiments still more complete, and give a greater similarity to the actual circumstances of a fire-box, Mr. Fairbairn constructed two boxes, one of which

is represented in section and elevation in the woodcut. The boxes were 22 inches square, and 3 inches deep; one plate being of copper, ½ an inch thick, the other, wrought iron, ⅜ths of an inch thick; these were riveted upon a frame of iron: the boxes being identical, except that one had the transverse stays placed in squares of 5 inches in the side (nearly the same as in the boiler above mentioned, which had exploded), the other in squares of 4 inches in the side. Water was forced in as in an hydraulic press by force-pumps; the stays in the former case having to support the pressure upon 25 square inches, in the latter only 16 square inches. It may be well to state to students that boilers are never tested by a steam-pressure, but by water-pressure from pumps, same as used for hydraulic presses; the experimenter may stand by with safety in the latter case, but, if steam were used, the effects would be like the bursting of a shell. Boilers must be proved under steam with the pressure they are intended to work at, to try if they are steam-tight, and the riveting and caulking good.



The model with stays 5 inches apart finally gave way by the head of the central stay being drawn through the copper plate, which, from its ductility, offered less resistance to pressure in that part where the stay was inserted, the pressure being 815 lbs. per square inch.

50. The other model, which is that represented in the woodcut, with stays 4 inches apart, gave way at 1625 lbs. pressure per square inch, failing by one of the stays stripping or tearing asunder the threads or screws of the iron plate at the end of the stay marked *a* in the woodcut, on the dotted line through which the section is taken.

The actual strain on each stay in tons is found as follows:— In the first case with 815 lbs. per square inch, and squares of 25 square inches depending upon each stay, we have

$\frac{815 \times 25}{2240} = 9$ tons. In the second case; $\frac{1625 \times 16}{2240} = 11\frac{1}{2}$ tons.

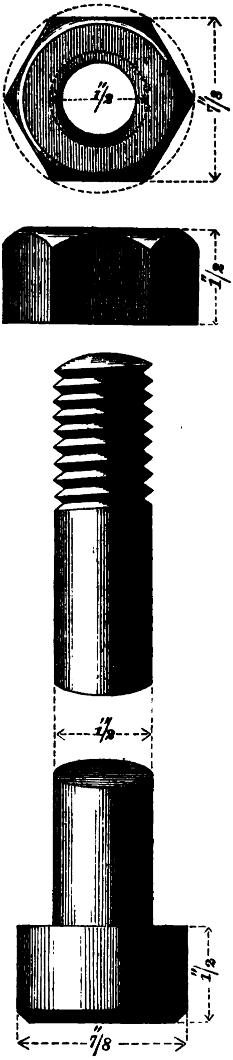
As in the latter case the diameter of the stay was $\frac{13}{16}$ths of an inch, the strain per square inch of section was about 13 tons, which is within the limits of rupture of wrought iron to a tensile force.

51. Still further to elucidate this subject, and bring the experiments to an identity with the circumstances of actual construction, Mr. Fairbairn obtained a locomotive engine, a fac-simile of that which had exploded, constructed by the same makers, and of the same age, having been at work from 1840 to the date of this experiment, 4th May, 1853, and having run about the same number of miles, namely 105,000. It was subjected to hydraulic pressure up to 207.5 lbs. per square inch; with this pressure one of the bolts of the cross-bar, Fig. 9, over the fire-box broke, which caused the experiment to be discontinued, as the leakage was greater than the force-pump could supply.

At 110 lbs. per square inch, this leakage had commenced, and gradually increased up to that of 207.5 lbs. The fire-box is described as having been, prior to the commencement of the experiment, considerably sunk or bulged, and the rivets as well as the stays much weakened. It may be concluded, therefore, that the engine which had exploded must have done so under a pressure of not less than 300 to 350 lbs. per square inch; the experiment having not in effect measured the ultimate resistance of the fire-box, but rather having been brought to a conclusion by what may be looked upon as the accidental failure of one of the bolts connecting the crown-bars to the roof.

52. In assigning the probable cause for the explosion of the engine, it had been objected that it was not possible the steam could have been raised from 60 lbs. per square inch, to that pressure of about 300 lbs. per square inch—which the above experiments would indicate as requisite to that effect—in the interval which elapsed between the safety-valve being screwed down, and the occurrence of the accident. An experiment was therefore undertaken to determine the rate of increase of pressure, temperature of steam, &c., in a locomotive engine, with the safety-valve screwed down, and the fire under the boiler. It resulted that in 22 minutes the pressure rose from 11.75 lbs. to 111.75 lbs. per square inch. From the general discussion of the law of increase of pressure and temperature, as exhibited in this experiment, and the formula thence derived, it appears that to raise the pressure from 60 lbs., as a commencing point, up to 300 lbs., would require but 28 minutes ("Useful Inform. for Engineers," Appendix. pp. 64–67).

53. Not less important as a method of combining into one structure several separate pieces, not only of wrought iron, but also of cast iron, is the wrought-iron screw-bolt and nut. It is but a few years since manufacturers of machinery have adopted any uniform system of proportioning the depth and diameter of the head and nut to the diameter of the blank shaft of the bolt, or assigning the due number of turns of the screw per inch, and proper depth of thread in connexion with that same unit of dimensions.



The woodcut represents at full size a $\frac{1}{2}$-inch bolt with head, nut, and screw; it will be observed at once that the depth of the nut and also of the head are exactly equal to the diameter of the bolt, and this holds good with all diameters of bolt: with a 2-inch bolt they are two inches, and with a 6-inch bolt they are six inches also, i. e. when the bolt is so used in any structure as to be resisting tensile forces; if, however, it be applied merely as a means of connecting separate parts, and by that duty not exposed to tensile strains, then the depths of the head and nut are only $\frac{3}{4}$ths of the diameter of the shaft of the bolt.

The diameter of the circle circumscribing the nut is about double the diameter of the blank shaft of the bolt, and, as the nuts are hexagonal, the side of the polygon will be nearly equal to that diameter. The external dimension of the nut, as generally given on the published drawings of the series of bolts, extending from 2 inches to $\frac{1}{4}$th of an inch in diameter of bolt, is that between two opposite parallel faces of the hexagon.

In the following Table the bolts differ in diameter by $\frac{1}{8}$th of an inch in the larger sizes, down to $\frac{1}{2}$-inch bolts; below that, by $\frac{1}{16}$th of an inch.

The breadth of nut differs by $\frac{3}{16}$ths in nearly every case.

Diameter of Bolt,	2	$1\frac{7}{8}$	$1\frac{3}{4}$	$1\frac{5}{8}$	$1\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{1}{4}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$
Breadth of Nut,	$3\frac{3}{16}$	3	$2\frac{3}{4}$	$2\frac{9}{16}$	$2\frac{3}{8}$	$2\frac{3}{16}$	2	$1\frac{13}{16}$	$1\frac{5}{8}$	$1\frac{7}{16}$	$1\frac{1}{4}$	$1\frac{1}{16}$	$\frac{7}{8}$	$\frac{25}{32}$	$\frac{11}{16}$	$\frac{19}{32}$	$\frac{1}{4}$
No. of threads per inch,	$4\frac{1}{2}$	$4\frac{1}{2}$	5	5	6	6	7	7	8	9	10	11	12	14	16	18	20

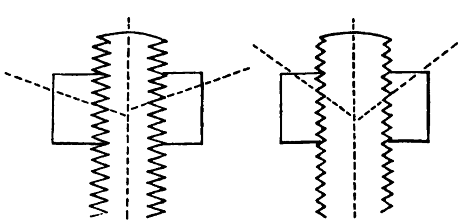
The following Table gives the number of threads per inch up to 6 inches diameter.

Diameter of Bolt, . .	$3\frac{1}{4}$	$3\frac{1}{2}$	$3\frac{3}{4}$	4	$4\frac{1}{2}$	$4\frac{3}{4}$	5	$5\frac{1}{4}$	$5\frac{1}{2}$	$5\frac{3}{4}$	6
No. of threads per inch,	$3\frac{1}{4}$	$3\frac{1}{4}$	3	3	$2\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{5}{8}$	$2\frac{1}{2}$	$2\frac{1}{2}$

54. If we conceive the transverse section of the bolt to be of such strength as that, on sufficient weight being applied longitudinally to it, the bolt may be drawn through the head, shearing the material along a cylindrical surface, which is the continuation of that of the bolt,—and, therefore, equal to the depth of the head multiplied into the circumference of the

bolt—then, by making the depth of the head equal to the diameter of the bolt (as appears above to be the practice), we have this area equal to four times the transverse area of the bolt, for the latter is expressed generally by $d^2 \times 3.14159 \div 4$, and the cylindrical surface resisting the shearing force is—depth $\times d \times 3.14159$; which last, when depth $= d$, is evidently 4 times the other: and as we know that the shearing resistance of any surface is equal to the transverse resistance of the same, ample strength appears to be secured by the proportions in use in construction. The force required to punch a hole 1 inch diameter in a ¾-inch plate is 46 tons, or 19.6 tons per square inch. To draw a ½-inch bolt through its head would thus require 15½ tons.

As to the nut and screw, we may conceive the transverse section of the bolt to be of such strength that (sufficient force being applied) it may either burst the nut, or strip the thread from the nut, or leave the threads of the bolt itself in those of the nut; in the second case the cylindrical surface has the base of the threads of the nut, and in the third the base of those of the bolt, each into the depth of the nut, for effective resisting area; the latter is evidently the lesser of the two, and as with the same diameter of bolt the outer cylindrical surface is constant, and the difference of radii of these two concentric cylindrical surfaces equal to the depth of the thread, we find that, the finer the screw, the stronger it is. Practical considerations, however, put a limit on the use to be made of this advantage, such as the greater wear and tear and liability to injury in screwing and unscrewing when the threads are of small depth, and the pitch fine.

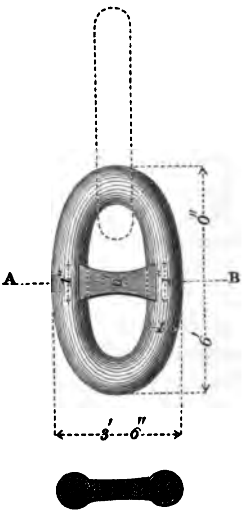


The force required to burst open a nut, and to resist which they must be of sufficient diameter, depends on the angle formed by the sides of the thread, which may be considered as parts of the surface of a cone, as in the dotted lines in the woodcuts, which are sections through the centre line of bolt and nut; the deep thread is drawn more obtuse, and the shallow thread more acute, throwing therefore a greater bursting strain on the nut. (Holtzappfel, vol. ii. pp. 655, 681).

The angle at which the sides of the thread of the screw intersect is generally 55° for all depths and pitch.

55. "The wrought-iron chain cable, which has now completely superseded those of hemp, was first brought into use about 1811, by Captain Sir S. Brown. The original cost is not much higher than the hempen cable. But the security afforded by it is vastly greater; for it is exposed to none of the deteriorating causes which render a hempen cable, after much use, comparatively so little trustworthy. The alternate wetting and drying, which saps the strength of a hempen cable, has no effect on one of iron. The friction against rocks, especially against coral, is often fatal to a hempen cable in a few minutes; but the same friction, after weeks of hard use, only slightly polishes a few links of the chain. In tropical countries, therefore, the introduction of chain cables has increased the security of ships at anchor ten-fold; but in every climate their advantage is immense. Nor does this superiority consist solely in their strength and durability, for they are managed with more facility, occupy far less space, and are stowed away with little, or, it may be said, no trouble at all; for as they are hove in, they fall quietly into and adjust themselves in a box or case near the hatchway, from which they are drawn up, when wanted, with comparatively small labour. The shackles, by which they are joined on to the anchors, are an immense saving of time in doing away with the labour of splicing and unsplicing, necessary when hempen cables were used."—(Captain B. Hall's *Voyages.*)

56. The woodcut represents in elevation the form and dimensions of a single common-link; and also a section through the link on the dotted line AB.



The diameter of the round bar out of which each link is formed is taken as unity, and then the longer diameter of the oval is made 6 units, the lesser 3.6; the *cast-iron* stay-pin is 0.6 in the centre, and 1 at the base, where it is shaped to the surface of the inner part of the bar. If then the diameter of the bolt, before being bent into the oval shape, and welded together, was 2 inches, we should from these proportions find the length 12 inches, the breadth 7.2 inches, and the stay-pin 2 inches at the base, and 1.2 in the central part.

The cable for a first-rate line of battle-ship is formed of links manufactured out of bar iron 2¼ inches in diameter; such a cable, 100 fathoms long, weighs, with 4 swivels and 8 joining shackles included, 12 tons 3 cwt., or 272 lbs. per fathom. That formed of bolt 1¼ inch in diameter weighs 84 lbs. per fathom, or 3 tons 15 cwt. for one hundred fathoms.

The large mooring chains for the buoys in the river Clyde are formed from bars 2⅝ths of an inch in diameter; each link weighs 42 lbs., and the coupling links from 50 to 52 lbs. The cables for the Great Eastern steam-ship are of unprecedented dimensions, each link weighing 70 lbs.

The 100 fathoms, or 600 feet, is divided into 8 lengths of 75 feet, or 12.5 fathoms, which are separately proved by an hydraulic press, with a table sufficiently long for one length of the cable to lie upon. The several lengths are united by joining shackles, and have one swivel in the middle of every other length. The proof—which is estimated at 630 lbs. per square ⅛th inch of transverse area—is, for a 2¼-inch, 91⅛ tons; for a 1¼-inch, 28¾ tons. The ultimate strength being about 125.9 tons and 38.5 tons respectively.

A hempen cable, 25 inches in *circumference* (for thus, and not by diameter, are they always specified) weighs 129.16 lbs. per fathom, or 5 tons 14½ cwt. per cable length of 100 fathoms, and by actual experiment one was broken with 103.25 tons. Further information is given in the Article "Cable" in "Aide Memoire of the Military Sciences," by Colonel R. J. Nelson, from which this section has been partly compiled.

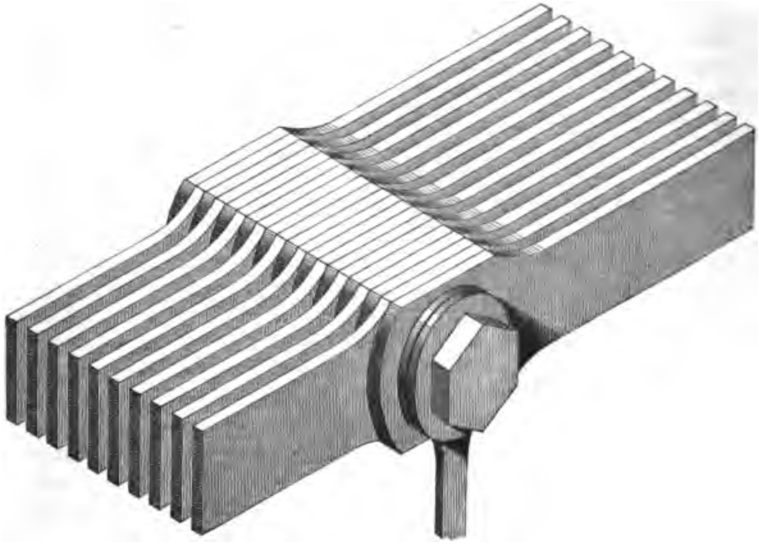
57. From experiments undertaken by Captain S. Brown in 1816, it appears that chains formed of round bolt 1½ inches in diameter gave, at a mean of 13 experiments, 76 tons for the ultimate strength of the double bolt—i. e. 38 tons for a single bolt 1½ inches in diameter, and as $(1.5)^2 \times 0.7854 = 1.767$ square inches, we have $\frac{38}{1.767} = 21\frac{1}{2}$ tons per square inch. The same machine gave the ultimate strength of bar iron at 25 tons; the ratio, therefore, of the strength of the simple bolt to the same, welded in the form of oval links of chain, is 100 to 86; these chains were, however, without the stay-pins, which are shown in the woodcut; these act under compression

as pillars in resisting the shortening of the smaller diameter of the oval of the link, and so also oppose the tendency of the longer diameter to become elongated under great strain, keeping the material better to its work, and inducing a higher ultimate resistance; the cable with stay pins being found of an ultimate strength very nearly equal to that of the iron in the simple bar form. The stay may thus be said to increase the strength about one-sixth part; the total weight is, however, increased in a ratio not much less, but the cost of the cast iron is far less than an equal weight of wrought iron, in this instance probably less than one-half. In end-links of a chain cable, which are necessarily without stay-pins, the strength is increased; the diameter of the bolt being 1.2, that of the common link being unity.

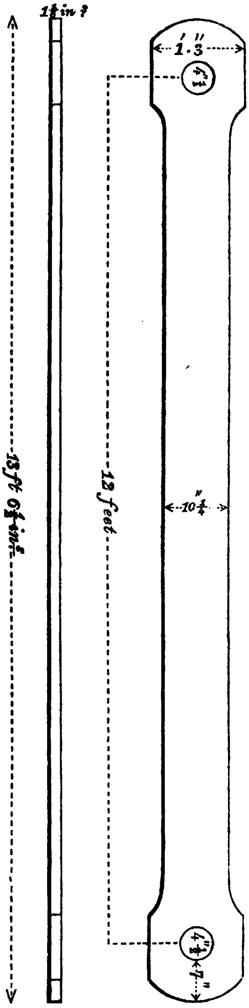
58. Another interesting and important application of wrought iron, under the head of resistance to direct tensile forces, has been in the construction of suspension bridges, by means of which a safe and permanent communication is secured in positions where it would have been impossible to erect a bridge of masonry, or, at least, so expensive as practically to amount to the same. In Great Britain the main-chains of suspension bridges are almost always formed of flat wrought-iron bar-links, arranged in sets, and united by transverse bolt-pins, through eyes drilled at each end On the Continent and in North America they are constructed of continuous wire ropes. The flat bar-links require to be manufactured with the utmost care, and by machinery of an expensive character, and are not so easily transported to remote and secluded districts, where it may be desirable to open up new lines of road, as the materials of a wire-rope bridge.

59. Fig. 10 represents one of the bar-links of the Menai Suspension Bridge, its side-link, and also an adjusting link, with its appropriate side-link. The transverse section in the line CD is 3.25 square inches, being 3.25 inches broad, by 1 inch thick, and 10 feet from centre to centre of bolt-hole; of these links there were 80, giving a total transverse area of 260 square inches, span 570 ft It is necessary to proportion the ends of the bar, so that the transverse section in the line AB be equal to that at CD, and also that the diameter of the bolt-pin be such that it may resist the shearing action of the tensile force to which the bars are subjected; and again, the interior surface of the eye on its extreme semi-circumference must be of area sufficient to resist any permanent alteration by the great com-

pression it is exposed to; and so of the surface of the transverse bolt-pin also. The bars in this Bridge were united by short side-links, 16 inches long, 8 inches broad, by one inch thick, and were thus laid in direct line:—Two bolt pins being required at each joint, the present practice dispenses with these, and lays the ends of the bars so far overlapping as to bring the eyes truly opposite, in which case a single bolt-pin only is required at each joint. The wood-engraving exhibits this method, showing in isometrical perspective the junction of the bars in one set of main-chains of the bridge over the Danube at Pesth, span 666 feet.



It is seen that there are alternately 10 and 11 bars in each set; nevertheless, the resultant of all the parallel forces of direct tension passes symmetrically through the centre line of each set, in one case lying in the middle bar of the 11, in the other in the middle space of the 10 bars. The vertical bar which supports the portion of the roadway from joint to joint is also shown, and the washer outside it, on which the hexagonal head of the bolt-pin rests, the opposite side having a similar vertical bar, washer, and hexagonal nut screwed upon the thread worked on one end of the transverse bolt-pin. The bars were all invariably 10.25 inches in depth, but varied in breadth from 1.08 to 1.125 inches for the sets of 11 bars, and from 1.186 to 1.25 inches for the sets of 10 bars, the lesser breadths in each set being at the centre of the main opening; the greater, near the top of the tower. The total area of transverse section would thus be about 486 and 510 square inches at those two extreme positions respectively, giving a difference of 24 square inches, lessening the weight of the main chains about 10 tons.



60. The form of the head of the bar in the Pesth Bridge is evidently superior to that of the Menai: in the latter, Fig. 10, the form of the outer line of the head is circular, concentric with the eye, and having apparently only in the line *ab*, transverse area equal to that of the bar itself at *cd*, being deficient in the requisite mass of iron. The lifting bar, § 63, is also superior, even at the end not stepped.

The bars of the Pesth Bridge, as in the wood-engraving, have the head formed by lines parallel to the lines of the bar itself, and we can readily conceive this head resisting any elongation, permanent or temporary, by that greater quantity of metal which it contains, external to a circle touching these right lines, and concentric to the eye.

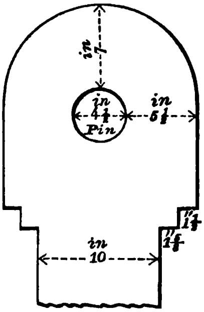
A special portion of this work will be devoted to the theory of the suspension bridge, so far as it may be needful to obtain rules, by calculation from mechanical principles, for the requisite dimensions of the several parts.

61. The Charing Cross Suspension Bridge—for foot passengers only—span 676.5 feet, has bar-links of 24 feet in length from centre to centre of the eyes, 7 inches in depth, and of a

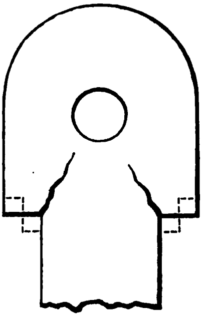
varying thickness, about 1 inch, those at the centre of the span being less in area than the more inclined bars at the summit of the towers: the total sectional area at the centre of the span being 296 square inches, and near the piers 312 square inches, being a difference of 16 square inches at these two points. The great length of the bars in this bridge diminishes very much the number of parts, and the number of surfaces to be truly fitted, and thus tends to secure each bar performing its intended duty.

The bars of the Chelsea Suspension Bridge are intended to be 16 feet 6 inches long, from centre to centre of eye or pin-hole.

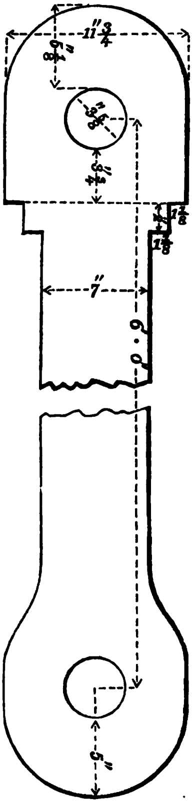
62. Mr. Tierney Clark designed an ingenious form of head for the lifting and retaining links of the Pesth Suspension Bridge, by which they were advantageously secured and raised. (Plate 22A, Part IV., Supplement to "Theory, Practice, &c. of Bridges:" J. Weale.)



The wood-engraving represents this form of head, which has underneath on each side double slotted steps, the projection of the upper 1½, the lower 1⅝ths of an inch, the vertical depth apparently the same. Clams or cheeks were constructed with corresponding slotted steps, by which the chains were gripped. The contrivance of the double step, instead of a single one of a double depth, obviates the liability to break by a fracture commencing at the re-entrant angle; the wood-engraving with the single step on each side, compared with that above, is sufficient to convince the judgment of this advantage; in fact, if from any want of true fitting the shoulder should in each case bear upon the clam on the points only, then the leverage to tear open the head at the angle is with the single step twice that which obtains in the case of the double-slotted steps, and the weight or pressure is also double, unless we suppose the workmanship of the double-step to be so extremely bad that the inner of the two does not bear its due portion of the load.



63. The plate-bar of the chains used in lifting the Conway and Britannia tubes is represented in the wood-engraving. These tubes were constructed on the shore, upon platforms of timber a little above the reach of the tide, and floated off at high water by pontoons let in under them, and then drawn into a position where they deposited the tubes, as the tide fell, upon a shelf of masonry at the foot of the piers which these tubes were destined to connect. They were elevated to the proper level of the rails by means of hydraulic presses at each end, placed sufficiently high in the piers to command the tubes at their greatest intended elevation. The plunger or ram carried a cross-head, from each end of which the chains, consisting of links of plate-bars, depended.



It will be observed that these plate-bars had the double-slotted steps underneath the head, similar to those in the lifting and retaining links of the Pesth Suspension Bridge; the slotted steps are here, however, differently and perhaps better proportioned, the depth of the step being 2 inches, and the projection 1⅛ inch.

Movable clams, Fig. 11, with corresponding steps were let in under the heads of the plate-bars, one set being placed on each end of the cross-head, and a second set upon the beams below, which supported the hydraulic press. These presses had an effective stroke of 6 feet, the same as the length of the links; when, therefore, they had elevated their load a lift of that height, it was necessary, in order to continue the operation of raising the tubes, to let down the ram and cross-head to the bottom of its stroke, and in the meantime retain the lifting chains and tube at the height attained; this was effected by closing the lower set of clams on the immediately contiguous slotted heads, and the water being now let out of the hydraulic presses, the weight of the tube was therefore resting on the lower set of clams, and the upper clams being opened released the top set of links, which were therefore removed. The cross-head having descended 6 feet, was yet a distance of 6 feet also from the lower clams placed on the beams that supported the press, and had therefore links in position ready to be gripped by the clams which it carried; these being now closed in, and water driven into the press, the ram and cross-head carrying its burthen was raised another 6 feet, the lower set of clams being opened when relieved of the weight; and thus the operation was continued until the required elevation was attained.—(Clark on "Britannia Bridge," pp. 604–614.)

A working model of this remarkable engineering operation, at a scale of $\frac{1}{16}$th or $\frac{3}{4}$ths of an inch to a foot, is in the Museum of this School of Engineering.

64. Let us examine the proportions of the several parts of these bars and pins:—First, as to the shearing resistance of the transverse section of the pin, compared with the tensile resistance of that of a plate-bar. From what has been proved in §41, we know that these areas may, for one shear, be equal.

The plate-bars were arranged in sets or links, alternately 8 and 9 in number, the former being 1.25, the latter 1.1 inches thick, compensating difference in number by difference in thickness, all having a common breadth of 7 inches, which gives a transverse section of 8.75 and 7.7 square inches for each bar respectively, or 70 square inches for the 8 bars, and 69.3 for the 9 bars.

Now the pin, being $3\frac{5}{8}$ inches in diameter, has a transverse section of 10.32 square inches to resist shearing, and as from Fig. 11 it is seen that there are 16 cross sections of the pin, giving 165.12 inches to resist shearing, and but 70 or 69.3 square inches, as mentioned above, to resist tensile strain—it is evident that there is a great excess of shearing resistance; but other circumstances forbid such reduction of the diameter of the pin as would have made these areas equal for.

Secondly, the sum of the areas of the slotted steps should be equal to the effective area of the inner and upper half surface of the eye of the head of the bar, each having to resist an equal compression arising from its due portion of the weight during the lifting; and the pin, being of the same diameter as the eye, has an equal surface under compression, a surface depending on the diameter, and thus the area of the slotted steps regulates the dimensions of the pin. The following calculation shows that the diameter, requisite to give effective area to resist compression, is greater than that to resist shearing, for the former area must equal the sum of the areas of the slotted steps. These being 4 in number in each bar, and projecting

1.25 inches, if we take one of the 8 plate-links 1.25 inches thick, we have for each bar 6.25 square inches, or 50 square inches for the 8 bars. The bearing surface of the pin—taking its whole upper semi-circumference as acting—is 7.12 square inches under each bar, or 57 square inches for all the 8 bars.

As the whole upper semi-circumference of the pin does not equally resist the vertical pressure upon it, the excess of 14 per cent. is, therefore, very judicious.

The compression amounted to 10 tons per square inch. The greatest tensile strain was 8.3 tons per square inch.—(Clarke on "Britannia Bridge," pp. 611, 612.)

It is probable that in lifting the tubes these chains were strained in a higher degree than ever practised before, but the extreme accuracy of workmanship prevented all unequal strain, and the tube was suspended on them for several weeks. The pin and bearing part of the pin-hole were visibly upset by the pressure. The depth immediately over the pin-hole, it may be observed, was greater than the width on either side of it.

65. Wrought iron in form of Wire Ropes has now gained extensive use for suspension bridges, railway inclines, mines, standing rigging, signal cord, &c., and has a much higher tenacity than common wrought iron, even of the best description.

For, in the first place, the iron out of which the wire is drawn must be of a very excellent quality in order to be at all suitable for that operation; and secondly, the additional tensile power of resistance it derives from this process of drawing into wire is nearly threefold. Thus, a rod of iron of dimensions capable of sustaining 1½ tons tensile force, when drawn out to three times the original length, or, which is the same thing, having only one-third of the original section, is still able to sustain 1½ tons; and the expense of this is about 30 per cent. of the original cost of the iron rod, but its length is increased 300 per cent., and this increase of length is equivalent to an increase of quantity in that ratio, as the strength is not diminished; hence the great economy of iron wire in those applications above enumerated.

It seems, however, not to retain in combination, when laid or twisted in spiral strands, that great superiority of tensile resistance over common wrought iron bars that it exhibits, as a single straight wire compared with the same section of wrought iron of ordinary manufacture. In this it is analogous to the yarns of a hempen cable, which lose nearly one-half of their strength, and one-third of their length when twisted, and forming part of a cable. It is easy to observe, when examining a cable of hemp or wire, that, when it is strained, all the fibres or wires cannot be equally resisting the tensile force, for they do not all make equal angles with the direction of the axis,—those, if any, which are parallel to the axis resist with the full power due to their transverse section, and the others less and less, as the angle of their inclination may be greater. And in wire cables for suspension bridges, which are mostly laid *parallel*, and bound round externally, and do not therefore come under the above disadvantage, it is yet nearly impossible, when laying the several single wires together, to secure each at an equal initial strain, so that when in use in the construction they may all perform an equal duty.

66. The following Table, published by Messrs. R. S. Newall and Co., exhibits the comparative dimensions, &c., of hemp and wire rope of equal strength. It will be observed that the transverse section of the hemp is nearly four times greater than that of the wire rope, and the weight double.

TABLE *showing the strength of Wire Rope.*

Hemp Rope.		Wire Rope, of equivalent Strength.				
Circumference.	lbs. weight per Fathom.	Circumference.	lbs. weight per Fathom.	Breaking Strain.	Working Load.	Price per Cwt.
3¾	4	1¾	2	3¾ Tons.	12 Cwt.	70*s*.
4½	5	2⅛	3	5½ ,,	18 ,,	68*s*.
5½	7	2¼	4	7½ ,,	24 ,,	66*s*.
6	9	2½	5	9 ,,	30 ,,	64*s*.
6½	10	2⅝	6	11 ,,	36 ,,	62*s*.
7	12	2⅞	7	13 ,,	42 ,,	60*s*.
7½	14	3⅛	8	15 ,,	48 ,,	58*s*.
8	16	3⅜	9	17 ,,	54 ,,	56*s*.
8½	18	3½	10	19 ,,	60 ,,	54*s*.
9	20	3¾	11	21 ,,	66 ,,	,,
9½	22	4	12	23 ,,	72 ,,	,,

Kuper and Co. have also published a Table for round and flat wire ropes compared with hempen ropes of equal strength, in which the numbers are nearly identical with those above.

67. In mines worked at the great depths they now are, especially in the coal-pits of the north of England,—where many shafts are about 1800 feet deep,—it is necessary that the mineral should be raised to the surface with great velocity

in order to obtain quantity sufficient to render the mine remunerative; any additional shafts for that purpose are, at these great depths, too expensive to be undertaken. The winding engine is therefore made of great power, and the rope of great strength; both the power and the strength being especially needed at the start, which they make nearly at full power, and almost immediately attain a high speed. For inclined planes on those parts of railways too steep for locomotive power, chiefly if not exclusively used on mineral lines, the wire rope, from its strength and durability, is found of the highest value; when used for this purpose, it is exposed to great friction, and sudden, severe strains. The drums around which it is wound are about 12 feet in diameter, much less than that being unsuitable, from the partial rigidity of the rope; and hence also it is only used for the standing rigging of vessels, for which it is very valuable, exposing so much smaller a surface to the action of the wind, stretching less than do hempen ropes, of much less weight for the same strength, and admitting of smaller spars being used.

68. The wire cable for suspension bridges has long been used for ordinary carriage roads on the Continent and in the United States, and very recently has had its application successfully extended in the latter country, so as to afford a nearly rigid way for the passage of railway trains.

The Niagara Suspension Bridge, designed by Mr. John Roebling, has a span of 821 feet, and a twofold way, the upper for the railroad, which is a single line; the lower for ordinary traffic: they are supported by four main cables of wire, each 10 inches in diameter, and each consisting of 3640 wires of small No. 9 gauge, 60 wires forming one square inch of solid section; hence we have 60.40 square inches of transverse section in each cable, the total amounting to 241.60 square inches, and the total number of wires being 14,560. The average ultimate strength of one wire is 1648 lbs.; it measured 18.31 feet per lb., and the specification required it to be tested thus:—Having been suspended between two posts 400 feet apart, it was drawn so that the deflection was 9 inches. From a number of trials, the results of which varied but slightly, it was found that the average deflection at which rupture took place was 0.683 feet, or little more than 8 inches. The above corresponds to about 1640 lbs. per single wire, or nearly 100,000 lbs. per square inch. It was also required that it should be bent square over the jaws of a large pair of pliers, and bear rebending without rupture. The greatest probable load on the railway and the common road below, together with that of the weight of the bridge itself, is calculated to produce a strain upon the cables one-fifth of their ultimate resistance. It is possible that a dense crowd of persons and heavy waggons on the lower road, simultaneously with a heavy train above, may give a higher value to this fraction, and greater strain upon the cables; but it is worthy of remark in this suspension bridge that it is, to a great extent, free from the danger arising from vibrations, whether vertical or lateral, which advantage is due partly to the great weight of the suspended structure compared with that of passing loads, as well as the triangular trussing introduced between the two roadways; and partly also to the ingenious arrangement of the main chains, which are towards the centre of the span brought within the vertical plane passing through the points of support on the towers, and the consequent inclined direction of the suspenders. Stays, from different points of the bank, are also attached to the roadway, both above and below the floor line. And again, underneath each rail of the railroad very deep girders of timber are placed, which spread the local weight over a great length of the structure.—("Papers, &c., on Public Works, British and American:" John Weale. 1856.)

This perfectly successful bridge proves practically that there is no inherent defect in the suspension principle, rendering it inapplicable to railway viaducts. For the opinions of eminent English engineers on the difficulty of using suspension bridges for railway traffic, consult E. Clark on the "Britannia Bridge," vol. i. pp. 38–41, 47, and 51. It is not improbable, however, that some modification of this mode of spanning very wide openings may be yet adopted in these countries, and made sufficiently rigid for railway trains, the great economy it offers by having so much of the material in tension being a strong inducement. The relative advantages of wire rope and plate-bars are given in Drewry on "Suspension Bridges," Section vi. p. 150.

69. Wrought iron is, in the progress of the constructive arts, continually superseding other materials, and as, in many of these applications, it would not be possible to classify them under the Practical Examples either of resistance to Direct Tension, Direct Compression, Transverse Strain, or Torsion, it may be instructive here to notice a few of them.

The framing of marine steam-engines has been latterly in a great degree constructed of wrought iron, and this, with other improvements in the boilers and engines, has enabled manufacturers to reduce the total weight, so that now the machinery weighs but 11 cwt. per horse-power, whereas formerly one ton per H. P. was the usual weight. This enables them to place much more powerful engines in vessels intended for speed and passenger traffic, the diagonal bracing of the hull by wrought-iron bars giving it strength sufficient to carry engines of immense power; and, on the other hand, if speed be not the object, we obtain from the lighter engines of the same power greater capacity for either cargo or coals.

70. The introduction of wrought-iron tanks into the navy and merchant service has in many ships doubled, and in all very greatly increased, the quantity of water which can be carried to sea; while the quality has been also improved in a very high degree. The following statement shows, in a rough way, the gain of stowage.

The cask called a butt occupies about 35 cubic feet of space in a ship's hold; and this space, if it were entirely filled with water, would contain 217 gallons. A butt, however, actually holds only 108 gallons; so that one-half of the space is lost by the thickness and form of the cask, which is the worst possible for stowage so far as room is concerned. An iron tank, of the kind chiefly used at present by ships of war, is called a four-feet cube, though it measures an inch more in its external dimensions, and occupies 68 cubic feet of space. This area, if entirely filled with water, would contain 424 gallons; but it actually holds 400, which is only 24 gallons less than the whole space occupied by the tank could possibly contain. In other words, by using casks, *eight-seventeenths*, 47 per cent., or nearly a half of the stowage-room, is lost; whilst by using tanks, only *one-seventeenth*, which is about 6 per cent., is lost.

A further increase in the stowage of water is gained by having smaller iron tanks, made either of a wedge or of a flat form, to suit the curve of the ship's hold, or to enter spaces too low to receive cubical tanks; of these there are twelve different sizes. As an example of the benefit of using wrought-iron tanks instead of casks at sea :—a 74-gun ship, when fitted with casks, stowed 47,624 gallons of water, or 212½ tons. But when iron tanks were substituted, she stowed 88,232 gallons, or 394 tons, or about ⅘ths more water. If we suppose her crew to number 600 men, at the full allowance of a gallon of water each per day, she could only remain at sea 80 days, or not quite *three* months, if it were kept in casks, but if the water were stowed in iron tanks, she might, with the same allowance, remain out 147 days, or very nearly *five* months.

This material is also now sometimes used in lieu of cast iron for pipes in shafts of mines, and obviates the liability to fracture of cast iron, and may be of much less weight, with great facilities of fixing and connecting.

All these applications mentioned in this section may be referred to the power of joining by riveting, and shaping the several parts to be connected, which wrought iron possesses, due to its ductility and toughness. To which properties we owe its adaptation to iron ship-building, one of the most important of its modern applications; but in this either the builders, the regulations of the Board of Trade, or Lloyd's, are greatly in fault, by following out in wrought iron the method of construction adopted in timber vessels. We shall, under the head of Direct Cohesion of Timber, find that though it has a very high tensile resistance in the direction of the fibre, its lateral resistance is very small, being in fir but one-twentieth of the former; hence the necessity of ribs to hold the planking together laterally, and keel and keelson on which to set up the ribs; but wrought iron is, as shown in § 31, nearly equally strong in every direction, a quality that should supersede the necessity of ribs and keel, which add so materially to the cost, without any corresponding advantage. The constructors of the Great Eastern steam-ship seem to have avoided these errors, in omitting both ribs and keel.

The large water-wheels for mill-power, constructed of wrought iron, in lieu of timber, may be mentioned as very important applications.

71. The specific gravity of wrought iron is about 7.6, which gives 475 lbs. per cubic foot; it is, therefore, very nearly equal to that of cast iron, which has been given, in § 11, p. 7, at 7.207, or approximately, 7.25,—wrought iron being thus about one-twentieth, or 5 per cent. heavier than cast.

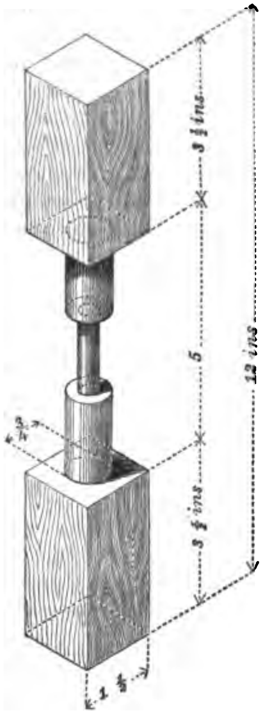
Hence, a bar, 1 square inch in section, and 1 foot in length, weighs 3.3 lbs., and therefore 1 yard long of same section weighs 10 lbs., *q. p.* As the rails used on railways, and angle-irons, are, as to their weight, always spoken of at so many pounds per yard, we can hence, from knowing the weight, always at once determine the area of the section, and *vice versâ*. Thus a rail weighing 90 lbs. per yard has 9 square inches of section; 75 lbs., 7½ square inches; 36 lbs., 3.6 square inches. Another convenient rule for the weight of plates is for every ⅛th of an inch of thickness the weight is 5 lbs. per square foot. This is, of course, identical with the former, for a square foot, 1 inch thick, consists of 12 bars, 1 foot long, with a transverse section of 1 square inch; hence, the weight is 12 × 3.3 = 39.6 lbs., *q.p.* 40 lbs. per square foot, the eighth part being 5 lbs. A cubic foot of wrought iron weighs 480 lbs., when estimated at 10 lbs. per yard for the inch square section.

DIRECT RESISTANCE OF TIMBER TO TENSILE FORCES.

72. **Timber.**—*The direct resistance of Fir Timber to a tensile force is about 5½ tons per square inch in carefully selected specimens.*

On this important material, so extensively used in construction, we have recorded the results of very numerous experiments, both on the Continent and in England; but it is to Barlow that we owe the most correct and consistent results. The peculiar character of timber requires, in the specimen under trial, and in the apparatus itself—as in cast and wrought iron—a special form and adaptation, in order to avoid erroneous results. In this Mr. Barlow has shown great skill. Until his experiments were undertaken, the form of the piece under trial seems to have been a long strip of a square transverse section, without any special contrivance to insure the coincidence of the line of direction of the external tensile force with the axis of the specimen under trial, a circumstance as important in experiments on timber as in any other material.

The form adopted by Mr. Barlow for the pieces under trial is shown in the woodcut in isometrical projection. A bar, 12 inches long, and 1½ inch square, is turned down in the lathe for a length of 5 inches to a cylinder ⅜ths of an inch in diameter, leaving 3½ inches at each end of the original rectangular form, and having very true shoulders in planes perpendicular to the axis of the cylindrical portion, to the lower of which surfaces the tensile force is applied; and on the upper the reacting support, by means of the apparatus shown in Figs. 12, &c., drawn from that in the model-room of this School, and which has been, to a great degree, constructed from the drawings given in Mr. Barlow's work. The middle part of this cylinder was reduced to a lesser concentric cylinder, about one-third or one-fourth of an inch in diameter, and is, therefore, the part at which the separation of the fibres must take place when the tensile force is applied.



73. Fig. 12 represents, in isometrical projection, the double wrought-iron beam, on the central circular raised surface of which, *aa*, rests the upper shoulder of the specimen, the cylindrical portion passing down through the opening, *b*, which is drilled of the same diameter as it; this is exhibited in Fig. 13, in which one-half of the double beam, with the connecting screw-bolts, is removed. The surfaces in which the parts of the double beam are in contact were truly planed and fitted, and the top and bottom edges, with the central raised portion of the top, were planed square to them; also the central orifice was drilled perpendicular to the plane of the upper edge of the double beam; when, therefore, it was set horizontal by a spirit level placed on the top surface, the round orifice, and consequently the specimen piece of wood it contained, were accurately vertical, and the shoulder rested solidly upon the central and now horizontal part of the top.

The mode in which the weight was applied is now easily understood. A cast-iron box, Fig. 14, in two symmetrical portions, which had each a semicircular orifice at the top, and a rectangular recess within, whose upper and inner surface was made truly perpendicular to the axis of its cylindrical orifice—accurately fitting, the former the larger part of the cylinder, and the latter the square part of the head of the piece to be experimented upon. The surfaces of contact were planed, and dowel-pins upon one-half fitted into corresponding holes in the other, so as to insure the perfect junction of the two parts, which, when closed upon the specimen, closely fitted the cylinder, and bore evenly upon the lower shoulder of the piece of timber, just as in the double beam of wrought iron mentioned above. To bind the two parts together, square bolts were passed through holes of the same size, and a split key, of wedge form, driven through a slot in the bolt, tightened all up, and the split being opened, it was thus secured from becoming loose. The only further arrangement needed is, that the shackle-bolt of the scale-pan carrying the weights be fixed in the central line. For this, a loop of iron is attached to the lower end of each part of the cast-iron box, so that they meet in this line, and at the same level. The pin-bolt of the shackle being passed through these loops, and the scale-pan being prevented from torsion or vibration by guides at the lower angles, all the requisites for a good experiment have been secured, and the direction of the tensile force must necessarily pass through the axis of the specimen.

74. *The ultimate resistance of Fir Timber to a direct tensile force is about 5½ tons per square inch.*

EXPERIMENTAL PROOFS.

TABLE *exhibiting the result of Experiments given by* MR. BARLOW *on this material.*

	1	2	3	4	5
	Specific Gravity.	Area in sq. inches.	Weight applied in lbs.	Weight per sq. inch. lbs.	Direct Cohesion. Average per square inch.
I.	600	0.088	1140	12933	lbs. 12857
II.	,,	0.096	1260	13073	
III.	,,	0.096	1191	12037	
IV.	,,	0.088	1160	13220	
V.	,,	0.098	1213	12371	
VI.	,,	0.088	1180	13448	
VII.	581	0.096	1059	11000	11549
VIII.	564	0.096	1201	12472	
IX.	601	0.096	1094	11360	
X.	611	0.096	1130	11736	
XI.	532	0.096	1076	11180	
XII.	590	0.096	1112	11548	

Adding the two means given in column 5, and dividing by 2240, we have, as above, 5.42 tons.

The first six experiments were made upon pieces, all cut from one plank, remarkably free from knots and irregularities, giving very high and uniform results. No. VII. broke by a part of the fibres drawing out of the head of the piece. No. IX. broke by the whole of the middle central cylinder drawing out of the head, to the length of about 2 inches, where there was a knot, which might break off the continuity of the fibres: this circumstance suggested to Mr. Barlow the experiments on lateral resistance given in § 77. In order to determine very exactly the area of the small central cylinder, a fine silk thread was wound round it ten times, and dividing this length by ten (or the number of volutions), the mean circumference was determined; and hence the diameter and area of transverse section were calculated.

75. The following Table gives the result of experiments on other descriptions of timber, showing their power of ultimate resistance, in pounds and tons per square inch, to a tensile force, and placed in the order of their strength. The numbers are means derived from many experiments, and the nearest round numbers.

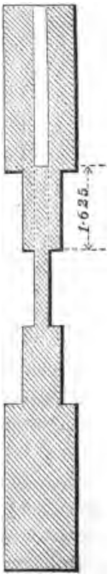
	lbs.	Tons.
Box, in round numbers, about	20,000	9.0
Ash, ,, ,,	17,000	7.6
Teak, ,, ,,	15,000	6.7
Fir, ,, ,,	12,000	5.5
Beech, ,, ,,	11,500	5.0
Oak, ,, ,,	10,000	4.5
Pear, ,, ,,	9,800	4.4
Mahogany, ,, ,,	8,000	3.6

That fir timber should have so much higher a place in the Table than oak or mahogany would not at first have been expected; but it is probable that their fibres are not so straight and free from twist as they are in fir timber.

76. Fir timber, and the other descriptions of wood also, are generally used in construction, either as pillars, struts, &c., or horizontal beams; but in some few cases we find its direct tensile resistance made use of in engineering construction, as in the pit-work of pumping-engines for unwatering mines; and here, as in other applications of timber, the real

difficulty is in the uniting the several portions together. These pump-rods are generally 10 inches square, and may be from 40 to 50 feet in length, halved together at the joint, as shown in Fig. 15, in isometrical projection, wrought-iron straps being laid on each of the four sides, extending equally above and below the joint; screw-bolts are passed through each of the opposite straps and the timber-rod, the holes in one pair of straps being arranged so as to alternate with those of the other pair. This combination may fail in one of three ways: either the timber-rod may be torn asunder; or the wrought-iron straps may fail about the joint, at a section through the bolt-holes of the straps; or, finally, the two first-named sections of the wood and the iron remaining uninjured, the failure may take place by the screw-bolts drawing out the pieces of timber before them, the lateral resistance of the fibres being insufficient to sustain the tensile force produced by the working resistance of the pumping. This last may be more clearly understood by Fig. 16, which shows in isometrical projection a single pair of straps connecting a butt-joint, which has failed by drawing out the piece of timber before the lowermost bolt, the lateral resistance of the fibres being overcome along the two surfaces, *a*, *b*, *c*, *d*, and *a*′, *b*′, *c*′, *d*′.

It is necessary, therefore, to determine experimentally the value of this lateral resistance, which manifestly affects the strength of this material in combination, and is, therefore, as important as the transverse resistance of the fibres, and bears some analogy to the shearing resistance of wrought iron, which in like manner became an indispensable datum when its strength in combination became necessary to be determined.



77. The same apparatus and form of specimen is adapted, with slight modification, to determine the lateral resistance of the fibres of timber, and was suggested to Mr. Barlow by some of the pieces, under trial for the transverse resisting power of the fibre, yielding, by drawing out either wholly or in part from the head of the piece, instead of breaking across. The head of a specimen was bored down, as shown in the woodcut (which is a vertical section through the centre line), until a length of only 1.625 inches was left between the bottom of this hole

and the central small cylinder. The several pieces were then placed vertically in the machine above described, § 73, and the weights laid on until the smaller cylinder was drawn out. The following Table (which is derived from experiments by Mr. Barlow) shows the amount of lateral adhesion of the fibres in fir timber in pounds per square inch, and gives the relation between it and the direct cohesion of the fibre itself, pointing out the proper length of the straps of wrought iron on each side of the joint, so that the lateral resistance of the surfaces of the parts liable to be drawn out by the through screw-bolts may not be less than that of either the direct cohesion of the pump-rods, or the direct cohesion of the wrought-iron straps, and that thus all three may be on the point of yielding simultaneously.

78. TABLE *showing the Lateral Cohesion of Fir Timber.*

	1	2	3	4	5
	Length drawn out.	Circumference.	Weight put on in lbs.	Weight reduced to one inch of surface.	Mean value of lateral cohesion on one inch of surface.
I.	1.625	1.10	996	556	lbs. 592
II.	1.625	1.15	1187	621	
III.	1.625	1.15	1117	584	
IV.	1.375	1.15	1066	634	
V.	1.500	1.15	1000	578	
VI.	1.500	1.15	1000	578	

79. Hence it appears that the lateral resistance is but one-twentieth part of the direct cohesive resistance of the fibres to a tensile force, and, therefore, the area of the portions liable to be drawn out should be twenty times greater than the transverse area of the pump-rod, working vertically, in order that it may be as efficient as the transverse section, and not yield before it. As an example, let us suppose the pump-rod 10 inches square, and having, therefore, 100 square inches in its transverse section, from which (if the through screw-bolts be 1 inch in diameter) we must deduct 10 square inches, giving the weakest transverse section at 90 square inches: the extreme bolts of the iron straps connecting the several rods must extend on each side of the joint so far from it, that, if they draw out the

pieces before them, as in Fig. 16, the two equal surfaces so separated must, for equal resisting power, be of a total area of 1800 square inches, or twenty times the effective transverse area of the rod (20 × 90 square inches); each surface therefore should be 900 square inches, and as the breadth is 10 inches, the length must be 90 inches, so that the strap should have a length of about 8 feet on each side both above and below the joint.

The effective transverse area of the iron straps, at the bolt-holes nearest the joint, must be rather more than one-fourth that of the timber rods they unite, that is, inversely as the ultimate resisting power of each material to a tensile force; these straps decrease in thickness towards the end, as in Fig. 18.

To resist the shearing of the bolts, their number must be such as to give a transverse area equal to that of the straps. In actual practice there are straps on each face of the rod, but for simplicity of illustration it has been supposed above that one pair only is applied.

80. The model of the Cornish Pumping Engine in the Museum of this School has been constructed at the scale of $\frac{1}{12}$. Figs. 17, 18 represent a portion of the pit-work. The main rod—which is composed of balks of the best Memel timber, perfectly sound and straight, and without knots or faults, such as are used for the masts of vessels, and as great in length as they can be obtained—is 14 inches square in the upper, decreasing to 12 in the lower part of the shaft. The upper iron straps are 18 feet long, 7 inches broad, and about 1¾ inches thick at the joint, tapering down to about ¾ inch at the end, having 17 through bolts which are a little more than 1 foot apart; the alternate straps on the other faces are set on the main rod lower than the former by half the interval between the bolts: those in the lower part of the shaft are 16 feet 10 inches long, with straps about 6 inches wide by 1¾ inch thick. As the several lengths of rods are halved into each other, the joint *rr* on the face does not occur at the middle of the strap.

81. The transverse section of the main rod, where 14 inches in the side, is 196 square inches, and 144 at the part 12 inches square; if the through bolts for each pair of straps be 1½ inches in diameter, we must deduct from the section of 196 square inches 40 square inches, leaving 156 as the effective section. The first-mentioned straps, § 80, have an effective transverse area of 38½ square inches at the line through the

bolt-hole nearest the joint (having made in the iron straps the corresponding deduction for the bolt-holes that was made in the timber), or at the weakest section. This is very nearly one-fourth of the timber main rod, which proportion corresponds to the relative ultimate tensile strength of the two materials, namely, 20 and $5\frac{1}{2}$ tons per square inch respectively.

The resistance of the bolts to shearing is no less amply provided for, as on either side of the joint there are, for each pair of the iron straps, 8 bolts $1\frac{1}{2}$ inch diameter, resisting at two surfaces each: we have $(32 \times 1.5^2 \times 0.7854 =)$ 56.55 square inches; and the shearing resistance per square inch being equal, *q.p.*, to the direct tensile, § 41, we should have the sum of the transverse areas of the four straps about equal to those of the bolts taken at two shears each.

The effective section of the main rod being 156 square inches, gives an ultimate tensile resistance of 858 tons, and that of the straps, estimated at $38\frac{1}{2}$ square inches, gives 770 tons: this is nearly 20 times greater than the actual average tension of the largest engines on the main rod in some of the deepest mines, which tension is required to be but a little more than the weight of the ascending column of water.

82. It is well here to state—though the pupils of this School study the action of this form of engine in the large model in the Museum—that in the Cornish pumping engine the principal part of the work is done by the descending stroke of the pump rods. When they are raised, the lowermost pump alone makes its lift, as it is found advantageous to have it an ordinary lifting or sucking pump; while in the others, which are forcing pumps, the water is merely drawn in under the plungers; but on the descent of the rods, all the lifts except the lowest raise their water by the action of these plungers in descending, and, by a due arrangement of valves, forcing this volume of water up the ascending pipes. The power by which this descent is effected is the weight of the rods, which in a deep shaft is very considerable; and it is therefore this weight, or at least a portion of it, which actually raises the water.

The raising of the rods is effected by the action of the steam pressing down the piston of the engine, and when this stroke is concluded, and the steam pressure removed, the rods acting by their weight depress the plungers, elevating the water by their descent, and are in the succeeding stroke again raised by the steam in its turn.

Thus the power of the elastic force of the steam is not immediately applied to raise the water, but lifts a dead weight, which on the return stroke becomes the moving power. This arrangement, necessarily arising from the use of the plunger pump, is peculiar to the Cornish system of pit-work.

The pump rods generally furnish more weight than is sufficient to overcome the pressure of the column of water in the ascension pipes, and the surplus is usually counterbalanced by weights attached to one end of levers or "balance bobs" placed at the surface of the ground, or in horizontal excavations in the shaft, the other end being connected to the main rod.

83. For example, in Davey's engine at the Consolidated Mines, the main rod is one-third of a mile in length (1760 ft.), and weighs 95 tons. The other rods weigh 40 tons, those, namely, parallel to the main rod, and strapped to it, that work the plunger poles and the lifting pumps at the bottom; total, 135 tons. But as the column of water in the vertical pipes weighs but 39 tons, only that weight, or a little more than it, is therefore required to raise the water; the remaining 96 are balanced by counter-weights, of which there are different kinds, namely, the "balance bobs" mentioned above, and the hydraulic balance.

The main rod has therefore in its descent, when forcing down the plungers, to resist a compression equal to the unbalanced pit-work, and for this duty it requires strength and dimensions sufficient to resist, as a pillar, its tendency to bend from the true vertical direction, the iron straps have in like manner to unite the several lengths of the timber pump rods with sufficient strength to obviate any tendency at the joints to deviate laterally, under this compression, from the true direction. This duty requires considerable strength in all the parts, perhaps even more than the former, that is, the ascending stroke.

The distribution of the wrought iron in the side straps, tapering from the greatest thickness in the centre less and less towards the end, is very judicious, whether we regard it as resisting tension in the ascent of the main rod, or as opposing the tendency to bend out of the true line in the compression of the descending stroke; in the former, it is evident that each successive bolt from the joint towards the end of the strap relieves all the remainder from a portion of the total burthen, thus admitting of a continually decreasing transverse section, and in the latter case, in resisting bending, the strongest section is no less required at the joint where the timbers are halved together. The main rod is of course guided in a true vertical line, in its up and down stroke, by pieces of timber firmly fixed in the shaft, and placed at sufficiently near intervals to resist the tendency to flexure by shortening the distances within which it may begin to develop itself.

84. The mineral solutions in the water of nearly all mines rapidly corrode whatever iron-work is placed in the shafts, and renders prudent a very large excess of strength in the iron straps; and again, the very irregular action that takes place at times in the working of the reciprocating machinery of the pump-work, compared with anything that occurs in uniformly rotating engines employed in manufactories, compels the use of a greater surplus strength, perhaps in this, than any other department of mechanical engineering. One among many accidents that may occur is the plunger, in its ascent, drawing air: at any one lift where this may take place, that plunger in its descent meets with no resistance, instead of having that of the pressure of the column of water in the lift above it to overcome.

85. And also the peculiar action of the Cornish pumping engine, which is both high pressure and condensing, requires great strength in the pit-work. The steam is admitted at a high pressure, above the piston, and after a short part of the stroke, from ⅛th to a half, is passed over, it is cut off, and that within the cylinder expands for the remainder of the stroke, and consequently continually diminishes in its elastic force.

Thus, suppose the cylinder to be 100 inches in diameter, the piston has, therefore, an area of 7854 square inches, and if the steam pressure be 20 lbs. per square inch, the total force depressing the piston amounts to 157,080 lbs., or 70 tons. If the stroke be 10 feet, and the expansion commence at ⅛th, this pressure will continue for 1 foot 3 inches; the pumps and pit-work being at opposite ends of the main beam, the descent of the piston elevates them, and if the beam had equal length of arms, the higher lengths of the rods would, for the first 15 inches of the ascent, have to resist a tensile force of 70 tons, and for the rest of the stroke continually less and less. On the introduction of this form, in lieu of Boulton and Watt's double-acting engine, the pit-work frequently gave way, and not until superior accuracy of construction and greater strength were adopted, did the full value of the improved engine become manifest.

86. Had the pump rods to be constructed and proportioned for tensile resistance only, a much less dimension would have sufficed, as we may see in Fig. 17, for the rod *m*, attached by the straps *n* to the main rod, is but 6 inches by 14; its duty is to connect the balance lever and the main rod. These "balance bobs" are adjusted to take off from the pit-work all the weight over and above the pressure of the column of water to be raised, and act always, both in the up and in the down stroke, in resistance to tension only. Each length of timber in the main rods is about 45 feet, this being the greatest average length to which the balks, as imported, run.

87. Some mining engineers make the iron strap yet wider, and place the bolts, as in Fig. 19, not in the same vertical line, which has evident advantages as giving greater resistance to any bending of the main rod at the joint in the plane of the strap, and also by engaging the lateral resistance of new lines of fibres of the timber.

88. We find in Simms' work on "Practical Tunnelling," pages 53 to 58, a description of an interesting and instructive application of timber under tensile resistance in supporting the brick-work of permanent and ventilating shafts, during the time in which that length of the tunnel, which is situated immediately underneath, is being built in and prepared for receiving the shaft permanently on the crown of the arch.

The brick-work of a shaft, 9 feet in diameter, forms a cylinder of great weight, as it contains 3 tons 7 cwt. to every yard in depth, and the friction of its outer circumference against the earth is not sufficient to resist its tendency to slide down, when the ground is removed from under it to construct the tunnel.

The shafts may be secured until they can be permanently united to the tunnel, either by supporting them below, or suspending them from above. The latter was adopted at Saltwood Tunnel, on account of the wet and porous sandy ground through which it was driven.

It is usual to make the hanging rods of iron, but the only available material at hand was larch timber. Fig. 20 shows a side view and front view of the hangings, and the method of joining or scarfing the pieces together to obtain the proper length. It is stated that in most of the shafts the larch rods stood the tensile force without exhibiting any apparent deficiency of strength; in one or two they appeared weak for the work; and in one case they broke, or rather tore away at the scarfing, but having shown previous indications of so doing, any casualty was stopped by timely propping.

89. The direct cohesion of larch timber, or the weight that one square inch would bear without being torn asunder, was taken at 9560 lbs. (= 4.27 tons), and as the section of each of the hanging rods was 9″ × 6″ = 54 square inches, these multiplied together give 516,240 lbs. = 230 tons, as the weight that would tear each rod asunder, and 460 tons for both.

The greatest weight of any shaft carried by both rods was 67 tons; the rods were then 6.8, or nearly 7 times stronger than their greatest load, that is, if the joint were as strong as the transverse section. This it does not seem to be, as the single pair of iron straps, 3 inches wide by 1¼ thick, are but 5 feet 4 inches long by scale; if we suppose the extreme bolts placed 2 feet 6 inches from the middle of the joint, we cannot have more than 1.25 square feet of surface on either side, resisting laterally being drawn out. The mode of halving shown in the side view, in which a key is driven in between the internal surfaces carved out of the solid wood of the end, gives nearly ⅓rd of a square foot; both amount to about 200 square inches, and, therefore, the sum of both rods, 400, equivalent to 20 square inches of transverse section applied in a direct resistance to a tensile force, as from §§ 78, 79, we find the lateral but the twentieth part of the longitudinal resistance. The joint is thus far weaker than the rod, the total resistance for the joints of both rods being only 85⅛ tons, but a little more than once and a fourth greater than 67 tons, the maximum load. Had the rods been square, 7.35″ in the side giving 54 square inches, instead of rectangular, 9″ × 6″, thus admitting of a pair of straps on the opposite faces, and if about 5 feet long on each side of the joint, we should have had it nearly as strong as the rod.

If iron had been used, four rods would have been employed, and if they had been constructed four times stronger than their greatest strain, each would have broken with 67 tons. At an ultimate tensile resistance of 20 tons to the square inch, the transverse area must have been 3.35 square inches; as a square rod the side would therefore be 1.83 inches, or 1⅚ths inch nearly; as a round rod the diameter would be 2.065 inches, or 2 inches and $\frac{1}{16}$th.

90. The top of the larch rods was well arranged: cleats were fastened on each face, forming underneath four shoulders resting on cross-pieces, supported by sills laid on the surface of the ground. The same arrangement below, the cleats being reversed, supports a strong frame on which the brick cylinder rested; in addition, the rod is carried down sufficiently to give support to a pair of struts which uphold the ends of a cross-tree, from the outer points of which short pieces depended, and thus afforded four points of support to the lower frame.

91. The king-post, Fig. 21, of the principal frame of a timber roof is acting against a tensile force, in holding up the tie beam at the centre. The queen-posts, Fig. 22, act in like manner, in a principal of that construction. A strap of wrought-iron, in both cases alike, passes under the tie beam, and is adjusted and tightened by an iron key and cotter passing through slots in the straps; a tenon is sometimes given to steady it in the tie beam.

The part within the dotted line is carved away to give a base for the rafter at the upper part and the strut below. The king-post of timber is now very frequently superseded by a wrought-iron bolt passing through a casting at top—shaped so as to receive the sloping rafters—and below, at the under surface of the tie beam, screwed against a washer plate. A cast-iron block, with a hole for the bolt, is placed on the top of the tie-beam to receive the ends of the strut. A wrought-iron bolt cannot be substituted for timber in any structure, unless that piece had been in tension. The tie-beam itself can be replaced by a wrought-iron bar, as is shown in the roof of the wharf-sheds at Liverpool, the foot of the principal rafter being now secured by a casting.

92. It is evident that the knowledge of the small lateral cohesion of the fibres of timber has influenced the practice of carpenters in the framing of the king-post and queen-post principals, as we observe the tie-beam always left much longer than the foot of the principal rafter; otherwise the tenon cut at the end of it would drive out the piece between the mortice and the end of the tie-beam; this additional length, however, is made use of to carry the pole-plate, which bears up the foot of each small rafter; and so also the top of the king-post is carried up as far as can be, and the upper part of it made use of to support the ridge-piece, and hold it in a vertical position; thus the pressure of the upper shoulder of the principal rafter, resolved vertically in the direction of the king-post, cannot force up the

piece above it, from its length;—the lower part of the king-post, forming an abutment for the struts, has like precautions. In the queen-post roof, though the upper ends of the queen-posts do not carry any ridge-piece, they are continued as far as the roof will allow. In some cases an iron strap, as at Fig. 21, takes off the pressure of the rafter which is resolved horizontally in the direction of the tie-beam.

These examples, to which the student with a cultivated power of observation can easily add, point out not only the great value of timber in construction, but the limits also to its utility from its weakness in the lateral cohesion of the fibres: this it is, evidently, that renders it so difficult to join together any two successive pieces in a structure or frame when resisting a tensile force. In § 70 the great superiority of the wrought-iron plate for ship-building is pointed out as depending, with other valuable qualities also, on its strength being equal in every direction,—the motion of the ship in rough weather requiring this peculiarity of the structure, that every part be prepared to resist equally in every direction. In timber vessels, the ribs at right angles to the longitudinal external planking and internal ceiling secure this object, which otherwise would depend on the lateral strength only of these last. An internal diagonal double-sheeting, crossed in opposite directions, in lieu of ribs, and effecting the same object, is now sometimes employed.

The value of timber in construction must rest upon its facility of conversion into any required form by cutting away, as in the king-post of a roof frame, the superfluous parts, either from the balk, as imported from America and the Baltic, which is seldom larger than 13 inches square, and from 40 to 60 feet long, or from lesser rectangular pieces sawn out from this, thus shaping out the form as designed.

It is evidently the lateral cohesion of the wood that gives strength to the common method of junction of pieces at right angles to each other, called dovetailing, and to some forms of scarfing and halving balks of timber, Fig. 23.

93. Carpenters, in framing together the ends of pieces of timber at right angles to each other, are well aware of its small lateral resistance, and leave the end of the piece that has the mortice much too long at the first, that the wedging up of the tenon may not drive out a part at the end, as explained in Fig. 24,—the dotted line showing the part to be sawn off when the frame is put together; and in window-sashes the same precaution is taken, the piece being carved and left

permanently. It is in vain, by complicated carving, such as may be seen in some old but now little used forms of halving and scarfing, to attempt to make any compound beam as strong in the joint as in the solid timber. Wrought-iron straps of suitable form and dimensions must receive and transmit the strains from one piece to another.

94. The weight of well-seasoned fir timber is about 31¼ lbs. per cubic foot, nearly the exact half that of water: and as wrought-iron is 475 lbs. per cubic foot, the ratio of their relative weights is 1 to 15.2. The ratio of their relative tensile strengths may be expressed nearly as 1 to 5.

Cast iron being about 450 lbs. per cubic foot, the ratio of the relative weights is 1 to 14.4, and that of the relative strengths is 1 to 1.273.

Hence we may express the relation of weight and strength in each of these three different bodies by one term or modulus, namely, the length of a rod of each of the same transverse section, which, being suspended vertically, will by its own weight overcome the tensile resistance at the point of support.

Let us suppose the transverse section for each to be one square inch: and first, in the case of fir timber, that it yields with 5½ tons, or in round numbers 12,000 lbs., per square inch, we have, therefore, 55,296 feet as the length of a uniform bar of this material; which, being suspended vertically, would tear asunder at the highest point.

In wrought-iron, the weight being taken at 10 lbs. per yard, we obtain in like manner 16,800 feet.

With cast iron, again, we have the weight 450 lbs. per cubic foot, and the tensile resistance 7 tons, or 15,680 lbs., per square inch; hence the length is 5018 feet.

Thus the relative numbers which combine the comparison of the weight and tensile strength of these three bodies are—

Fir timber, 55,296; Wrought-iron, 16,800; Cast-iron, 5018.

A rod of fir 16,800 ft. long would yet sustain, in addition to its own weight, a further charge of more than 8000 lbs. A wrought-iron bar, at that length, would tear asunder by its own weight: so that structures of wrought iron would reach their limit of size sooner than the same, if made of timber, as far as the parts in tension are concerned.

95. What are called bond timbers in brick buildings may be added as an instance of the use of timber in tension: they

are the thickness of a course of bricks, and carried across the openings of windows, &c.; latterly, hoop iron has been applied in lieu of wood to the same purpose,—that of binding together the structure while the mortar is yet fresh. Indeed, bars of wrought iron might have been instanced under its "Applications," as so used in large buildings of stone. Thus, the base of the dome of St. Paul's is bound round with several iron bars, keyed together in lengths, and of transverse dimensions, about 7 inches deep by 1½ broad.

DIRECT RESISTANCE OF CAST IRON, MALLEABLE IRON, AND TIMBER, TO COMPRESSING FORCES.

96. As in the experiments on the direct ultimate resistance of the fibres and particles of rigid bodies to tensile strains, we have found a force of cohesion opposing the separation of the parts, so now we find the same bodies resisting with energy all strains tending to compress the particles into closer union.

In almost every substance the amount of the ultimate resistance to compression is far higher than that to tension, nearly six times greater. Malleable iron, and some kinds of timber, form marked exceptions; in malleable iron it is even less, in a slight degree, than its tensile strength. No course of reasoning could have led us *a priori* to the knowledge of these facts; it is by experiments alone that they are discovered, and afterwards brought to bear on the art of construction.

The phenomena now met with are not, however, of that simple character we have hitherto observed under the head of resistance to tensile forces, in which it was found that the ultimate strength of any particular material depended (when the precautions mentioned in §§ 14 to 17 are complied with) on the area of the transverse section, and on that alone. On the contrary, the ultimate resistance to compression in each different body depends in a very marked degree on the form and proportions of the specimens under trial, which circumstances had no influence whatever in the former species of resistance.

97. To illustrate this, let us suppose two uniform straight bars of any material, lying on a horizontal surface, and, to fix the ideas, let us suppose them 5 feet long, and in section 1 inch square. If now a tensile force be applied to one, and a compressive force to the other; the line of direction of the force in each being supposed coincident with the axis of the bar. It

will be found that the bar opposing tension will resist with the full power due to its transverse section. But with the latter, as the compressive force is gradually increased, we perceive that the bar bends out of the rectilinear direction in which it had previously been lying, and in which the force is acting, and this bending becomes greater and greater as the applied force is increased, until finally it breaks across in a manner somewhat similar to what it would have done if under the action of a transverse force, and therefore, it only in a very small degree develops the resistance to compression of that material of which it is composed. Under the third head of transverse resistance it will be shown that a bar broken transversely must have one part of its cross section engaged in resistance to a tensile, and another part to a compressive force, reacting against the former, and so resisting less than if the whole area had been engaged in opposing the compressive force.

Let each bar be now shortened by one-half, the section remaining the same, and the force applied being as before. The tensile resistance depending, as it does, only on the transverse area, will be identical in both cases, but the resistance to compression, although the bar be found to bend as it did in the first instance when 5 feet long, and break in the same manner, will be far higher; and if these bars had been shortened again and again, we should have found the tensile resistance remaining constant, and the compressive continually augmenting: and, moreover, as the ratio of the length to the lateral dimension decreases, we find the appearance of bending gradually disappears altogether, and the crushing proper commences: the bars having passed through an intermediate state, in which they yielded partly by bending and partly by crushing: when the ratio of the height to the least lateral dimension is not less than 30 to 1, the pillar opposes the compressive force by its transverse strength alone; from that ratio to about, 5 to 1, it resists partly by its compressive and partly by its transverse strength; when about from 5 or 3 to 1, and under, the specimen yields only by being crushed. The manner in which this takes place varies with the different texture of the body, whether crystalline, fibrous, or laminated. The series of experiments on the direct resistance of bodies to compression naturally divides itself, therefore, into two distinct heads:—

98. 1st. Experiments on the resistance of bodies which are so short, compared with their lateral dimensions, as to be crushed with little or no flexure.

2nd. Those on the resistance of pillars long enough to break by flexure.

99. **Cast Iron.**— *The ultimate resistance of Cast Iron to a direct compressing force is about* 42 *tons per square inch, and is proportional to the area of transverse section in action.*

The machine by which Mr. Eaton Hodgkinson experimented is shown in isometrical projection in Fig. 25, and is ingeniously adapted to test the strength both of the short specimens and of long pillars.

The lever ABC is the same as that described in Fig. 5; it is here held down, at the end C, by the top D of the screw DEF, through a perforation in the end of which the lever passes, and is acted upon by an arris immediately below the point D, the bottom of the screw, near to F, being screwed down firmly into a cast-iron frame G. This frame is affixed firmly to a large cast-iron box IH, which passes, at its lower end, below H into the ground, in which it is firmly supported, laterally by masonry, and vertically by pillars, one of which is represented by KL. These pillars are employed to counteract the horizontal thrust in the direction B, caused by the weight laid on. The lever is prevented from acting in any other but a vertical position by means of the timbers, which are represented by MN. Between these vertical timbers the lever rises or falls without friction, and may be supported at any height by means of the wedges O, P. The lever, at a point below B, acts upon a cylindrical bolt, rounded at the top; this bolt, passing through the neck I of the large cast-iron box IH. The bolt here mentioned is made to pass easily through the box at I, but without lateral play, the top being rounded, that its centre only may be acted on by the lever, which is kept as nearly horizontal as possible; and to prevent the lever from being crushed at the point of pressure, a piece of steel, *i*, is attached to the bottom of the lever, and placed between it and the top of the bolt. The lower part of the bolt is made larger than the other parts, and turned flat and perpendicular to the axis, in order to press vertically upon a column placed under it.

The box is formed so as to admit of shelves S, of great thickness, placed at different heights, so that columns of any length, up to 10 feet or 11 feet, may be made to rest upon the shelves, raised to their proper height, and be acted upon at the top by the bolt passing through I, and pressed down by the lever. The shelves S, in all positions, whether higher or lower, are made (with great care) to be perfectly parallel; they are parallel, too, to the flat bottom of the vertical bolt, passing through I, so that a column, whether long or short, may be crushed between these surfaces, which are necessarily parallel. The utmost care having been taken to make the crushing surfaces parallel, other precautions were used in making the experiments. As, for instance, to prevent irregular action, from those surfaces being themselves crushed by the great weights employed, a small disk of hardened steel was placed under and over the specimen *m*. And, to prevent the specimen itself being injured in consequence of any unavoidable irregularity between it and the hard surface of the steel, a very thin sheet of lead, such as is used for lining tea-chests, was interposed between the ends of the specimen and the steel disks. This lead, filling up all interstices, acted well, and gave increased regularity to the results.

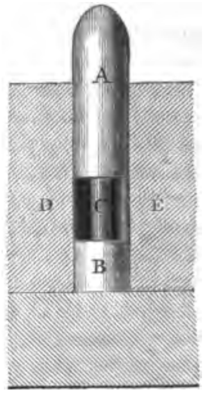
100. The specimens were first turned and squared with great care, and afterwards crushed between the two disks of hardened steel, which, from the nature of the apparatus employed, were necessarily parallel; and thus the resultant of the compressing force passed through the axis of the specimen, which will afterwards appear to be of the greatest importance.

All the specimens, unless marked otherwise, were from Low Moor iron, No. 1, and were cut out of the ends of the castings torn asunder, Table § 8, these ends being about 9½ inches broad and 1¾ inches thick.

101. In crushing the shorter specimens, it seems, from the description given by Mr. Eaton Hodgkinson at pp. 316, 317 of his "Experimental Researches on the Strength of Cast Iron," that in order to make the lever, during its descent, act uniformly upon all parts of the specimen, the body was crushed between two flat surfaces, kept perfectly parallel, and the ends of the prism themselves were turned parallel and at right angles to their axes, so that when the specimen was placed between the crushing surfaces, its ends might be completely bedded upon them. For this purpose a hole, 1¼ inch in diameter, was drilled through a block of cast iron about 5 or 6 inches square, and two steel bolts were made, which just fitted this hole, but passed easily through it; the shortest of these bolts was about 1¼ inches long, and the other about 5 inches; the ends of these bolts were hardened, having previously been

turned quite flat and perpendicular to their axes, except one end of the larger bolt, which was rounded.

The specimen was crushed between the flat ends of these bolts, which were kept parallel by the block of iron in which they were inserted. In the woodcut, A and B represent the bolts, with the prism or specimen between them, and DE the block of iron. During the experiment the block and bolt B rested upon a flat surface of iron, and the rounded end of the bolt A was pressed upon by the lever. There was another hole drilled through the block at right angles to that previously described; this was done in order that the specimen might be examined during the experiment, and previous to it, to see that it was properly bedded. The whole being placed upon the upper shelf of the box, the lever described above is made available for the experiment.



102. THE ULTIMATE RESISTANCE OF CAST IRON TO A COMPRESSIVE FORCE IS ABOUT 42 TONS PER SQUARE INCH.

Experimental Proofs.

TABLE *showing the Crushing Strength of Cast Iron of different forms of section, as Right Cylinders and Prisms, and of different magnitude, all cut out of the same Iron.*

The following Tables have been selected from the Appendix A of the Report on Iron in Railway Structures. They have been much less abridged than the Tables on Tensile Resistance, §§ 8 and 9, and it is hoped that so it may be instructive to Students as indicating the amount of accuracy and labour required in these investigations, and which their Author, Mr. E. Hodgkinson, has displayed in carrying them on; the fuller Tables also show the wide basis on which the results are founded, and the great number of distinct experiments required.

The iron chosen was that of Low Moor No. 1, and the experiments formed two series: the height of the specimens in one being intended to be equal to the diameter, or side of the base; and, in the other, to double that dimension. In both cases the height was so small that the specimen could not bend before crushing.

Before each experiment was commenced, a very thin sheet of lead was laid over and under the specimen, to prevent any small and unavoidable irregularity between its flat surfaces and those of the parallel steel disk between which it was to be crushed.

103. *Cylinders.*

Height of Specimen.	Diameter of Specimen.	Area of Section.	Crushing Weight of the Specimen.	Crushing Weight per Square Inch of Section.	Mean Crushing Weight per Square Inch of Section.
Inch.	Inch.	Sq. Inch.	lbs.	lbs.	
¼	.253	.0503	3143	62485	61627 lbs. =27.512 tons.
	.255	.0511	3159	61819	
	.253	.0503	3047	60576	
½	.251	.0495	3383	68343	65739 lbs. =29.347 tons.
	.253	.0503	3187	63359	
	.251	.0495	3243	65515	
½	.508	.2027	12848	63384	64337 lbs. =28.722 tons.
	.510	.2043	12988	63573	
	.511	.2051	13548	66055	
1	.508	.2027	13408	66147	66682 lbs. =29.769 tons.
	.511	.2051	13828	67420	
	.512	.2059	13688	66478	
¾	.752	.4441	30195	67991	64534 lbs. =28.810 tons.
	.756	.4489	28885	64346	
	.757	.4501	27575	61264	
1½	.758	.45126	24954	55298	56445 lbs. =25.199 tons.
	.753	.4453	25828	58000	
	.753	.4453	24954	56038	
1	1.004	.7917	49992	63145	62451 lbs. =27.880 tons.
	1.004	.7917	48681	61489	
	1.003	.7901	49555	62720	
2	1.002	.7885	44313	56199	55973 lbs. =24.988 tons.
	1.005	.7933	45187	56961	
	1.005	.7933	43440	54758	
2½	1.260	1.247	80288	64385	63639 lbs. =28.410 tons.
	1.261	1.249	78554	62893	

We are now enabled to perceive, not only the absolute amount of resistance per square inch, but also, to a great extent, the truth of the second part of the proposition, that the resistance is proportional to the area of transverse section, which will be deduced from this Table hereafter.

Three columns of the original Table have been omitted. They are: "Intended diameter and area of section;" "Crushing weight for the intended area of section;" "Mean crushing weight for the intended area of section." The former, it will be surmised from the second column of the Table, were 0.25, 0.50, 0.75, 1.00, 1.25 inches respectively, and from which they differed only in the place of thousandths, care having been taken, it would seem, to keep the specimens always a little in excess.

104. *Triangular Prisms.*

The base of each prism was intended to be an equilateral triangle, with sides 1 inch in length.

Height of Specimen.	Side of Base.	Height from Apex to Base.	Area of Section.	Crushing Weight of the Specimen.	Crushing Weight per Square Inch of Section.	Mean Crushing Weight per Square Inch of Section.
Inch.	Inch.	Inch.	Inch.	lbs.	lbs.	
1	1.006	.874	.4396	30632	69681	66988 lbs. =29.905 tons.
	1.005	.870	.4371	30196	69082	
	1.003	.870	.4363	27138	62200	
2	1.006	.873	.4391	30632	69760	69540 lbs. =31.045 tons.
	1.006	.866	.4356	30196	69320	

105. *Rectangular Prisms.*

The base of each prism was intended to be 1 square inch.

Height of Specimen.	Dimensions of Base.	Area of Section.	Crushing Weight of the Specimen.	Crushing Weight per Square Inch of Section.	Mean Crushing Weight per Square Inch of Section.
Inch.	Inch.	Inch.	lbs.	lbs.	
1	1.008 × 1008	1.016	62659	61672	62972 lbs. =28.112 tons.
	1.005 × 1000	1.005	61786	61478	
	1.005 × 10075	1.0125	66590	65766	
2	1.005 × 1005	1.010	59164	58578	59219 lbs. =26.437 tons.
	1.004 × 1007	1.011	56981	56361	
	1.003 × 1003	1.006	63096	62719	

NOTE.—The exact dimensions of the base of each cylinder, or prism, in the Table above, were obtained by careful admeasurement; but the heights were given as they are intended to be made, since a slight change in that would make no appreciable difference in the strength.

106. The average for the cylinders was 27.848 tons per square inch, for the prisms 28.875; but some earlier experiments gave a higher result for cylinders than for prisms. The average for all is 28.532.

107. TABLE *showing the Crushing Strength of Cast Irons from different parts of the kingdom.*

Results of experiments on the Crushing Strength of Cylinders of Cast Iron of various kinds; the Diameters of the Cylinders being turned to ¾ inch each, and the Heights being ¾ and 1½ inches respectively.

The experiments were made as in the last Table, and a very thin sheet of lead was put over and under the specimen, as before, to prevent irregularity of action during the experiment.

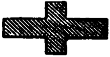
Description of the Iron.	Height of Specimen.	Diameter of Specimen.	Area of Section.	Crushing Weight of the Specimen.	Mean Crushing Weight per Square Inch of Section.
	Inch.	Inch.	Inch.	lbs.	General Mean.
Low Moor Iron, No. 1,	¾	.752	.4441	30195	60489 lbs. = 27.004 tons.
		.756	.4489	28885	
		.757	.4501	27575	
	1½	.758	.45126	24954	
		.753	.4453	25828	
		.753	.4453	24954	
Low Moor Iron, No. 2,	¾	.753	.4453	43736	95928 lbs. = 42.825 tons.
		.753	.4453	45483	
		.753	.4453	43736	
	1½	.753	.4453	40679	
		.753	.4453	41552	
Clyde Iron, No. 1, . .	¾	.764	.4584	41552	90805 lbs. = 40.537 tons.
		.764	.4584	41989	
		.764	.4584	44173	
	1½	.764	.4584	40679	
		.764	.4584	40242	
		.764	.4584	41116	
Clyde Iron, No. 2, . .	¾	.763	.4572	49852	106011 lbs. = 47.326 tons.
		.763	.4572	52035	
		.763	.4572	48978	
	1½	.763	.4572	47668	
		.763	4572	45920	
		.763	.4572	46357	
Clyde Iron, No. 3, . .	¾	.749	.4406	47668	106039 lbs. = 47.339 tons.
		.749	.4406	46794	
		.749	.4406	47231	
	1½	.749	.4406	45046	
		.749	.4406	48104	
		.749	.4406	45483	
Blaenavon Iron, No. 1,	¾	.753	.4453	41989	85710 lbs. = 38.263 tons.
		.753	.4453	38932	
	1½	.753	.4453	35874	
		.753	.4453	35874	
Blaenavon Iron, No. 2, 1st sample, . .	¾	.760	.4536	54656	110006 lbs. = 49.109 tons.
		.760	.4536	52036	
	1½	.761	.4548	47231	
		.761	.4548	45920	

Crushing Strength of Cast Irons—continued.

Description of the Iron.	Height of Specimen.	Diameter of Specimen.	Area of Section.	Crushing Weight of the Specimen.	Mean Crushing Weight per Square Inch of Section.
	Inch.	Inch.	Inch.	lbs.	General Mean. ‡
Blaenavon Iron, No. 2, 2nd sample, .	¾	·747	·4383	30195	68545 lbs. = 30.600 tons.
		·747	·4383	29759	
		·747	·4383	30195	
	1½	·749	·4406	32379	
		·749	·4406	28448	
		·749	·4406	29759	
Calder Iron, No. 1, .	¾	·763	·4572	33440	74088 lbs. = 33·075 tons.
		·763	·4572	32574	
	1½	·763	·4572	33008	
		·763	·4572	36471	
Coltness Iron, No. 3, .	¾	·774	·4705	47725	101005 lbs. = 45·091 tons.
		·773	·4693	46426	
	1½	·772	·4681	45920	
		·772	·4681	49415	
Brymbo Iron, No. 1, .	¾	·748	·4394	32989	75246 lbs. = 33·592 tons,
		·748	·4394	32380	
		·748	·4394	33253	
	1½	·748	·4394	33690	
		·748	·4394	32816	
Brymbo Iron, No. 3, .	¾	·749	·4406	33690	76545 lbs. = 34·171 tons.
		·749	·4406	33690	
		·749	·4406	33253	
	1½	·749	·4406	35000	
		·749	·4406	32816	
Bowling Iron, No. 2, .	¾	·749	·4406	32816	75058 lbs. = 33·508 tons.
		·749	·4406	33253	
		·749	·4406	34563	
	1½	·749	·4406	32379	
		·749	·4406	32816	
Ystalyfera Anthracite Iron, No. 2, . .	¾	·749	·4406	43299	97742 lbs. = 43·635 tons.
		·749	·4406	42863	
		·749	·4406	45920	
	1½	·748	·4394	41552	
		·748	·4394	42426	
Yniscedwyn Anthracite Iron, No. 1, .	¾	·747	·4383	36311	81084 lbs. = 36.198 tons.
		·747	·4683	36311	
		·747	·4383	37184	
	1½	·748	·4394	34563	
		·748	·4394	34563	
Yniscedwyn Anthracite Iron, No. 2, .	¾	·749	·4406	33253	76246 lbs. = 34·038 tons.
		·749	·4406	35000	
		·749	·4406	33690	
	1½	·747	·4383	32816	
		·747	·4383	33253	

108. TABLE *showing the Comparison of Tensile and Crushing Strengths in Cast Iron.*

This Table contains the absolute and comparative Strengths of different kinds of Cast Iron, to bear a Tensile and a Crushing Force, from the Results in Tables 8 and 107.

The specimens torn asunder had their sections in the form , and the crushed specimens had cylindrical sections turned to be ¾ inch diameter, the heights being ¾ and 1½ inches respectively.

Description of the Iron.	Tensile Strength.	Crushing Strength.		Ratio of Powers to resist Tension and Compression.	
	Tensile Strength per Square Inch of Section.	Height of Specimen.	Crushing Weight, per square Inch of Section.		
	Mean.	Inch.	Mean.		Mean.
Low Moor Iron, No. 1, . . .	12694 lbs. = 5.667 tons.	¾	64534 lbs. = 28.809 tons.	1 : 5.084	1 : 4.765
		1½	56445 lbs. = 25.198 tons.	1 : 4.446	
Low Moor Iron, No. 2, . . .	15458 lbs. = 6.901 tons.	¾	99525 lbs. = 44.430 tons.	1 : 6.438	1 : 6.205
		1½	92332 lbs. = 41.219 tons.	1 : 5.973	
Clyde Iron, No. 1,	16125 lbs. = 7.198 tons.	¾	92869 lbs. = 41.459 tons.	1 : 5.759	1 : 5.631
		1½	88741 lbs. = 39.616 tons.	1 : 5.503	
Clyde Iron, No. 2,	17807 lbs. = 7.949 tons.	¾	109992 lbs. = 49.103 tons.	1 : 6.177	1 : 5.953
		1½	102030 lbs. = 45.549 tons.	1 : 5.729	
Clyde Iron, No. 3,	23468 lbs. = 10.477 tons.	¾	107197 lbs. = 47.855 tons.	1 : 4.568	1 : 4.518
		1½	104881 lbs. = 46.821 tons.	1 : 4.469	
Blaenavon Iron, No. 1, . . .	13938 lbs. = 6.222 tons.	¾	90860 lbs. = 40.562 tons.	1 : 6.519	1 : 6.149
		1½	80561 lbs. = 35.964 tons.	1 : 5.780	
Blaenavon Iron, No. 2, 1st sample, ...	16724 lbs. = 7.466 tons.	¾	117605 lbs. = 52.502 tons.	1 : 7.032	1 : 6.577
		1½	102408 lbs. = 45.717 tons.	1 : 6.123	

Comparison of Tensile and Crushing Strengths in Cast Iron—continued.

Description of the Iron.	Tensile Strength.	Crushing Strength.		Ratio of Powers to resist Tension and Compression.	
	Tensile Strength per Square Inch of Section.	Height of Specimen.	Crushing Weight, per Square Inch of Section.		
	Mean.	Inch.	Mean.		Mean.
Blaenavon Iron, No. 2, 2nd sample, . . .	14291 lbs. = 6.380 tons.	¾	68559 lbs. = 30.606 tons.	1:4.797	1:4.796
		1½	68532 lbs. = 30.594 tons.	1:4.795	
Calder Iron, No. 1,	13735 lbs. = 6.131 tons.	¾	72193 lbs. = 32.229 tons.	1:5.256	1:5.394
		1½	75983 lbs. = 33.921 tons.	1:5.532	
Coltness Iron, No. 3,	15278 lbs. = 6.820 tons.	¾	100180 lbs. = 44.723 tons.	1:6.557	1:6.611
		1½	101831 lbs. = 45.460 tons.	1:6.665	
Brymbo Iron, No. 1,	14426 lbs. = 6.440 tons.	¾	74815 lbs. = 33.399 tons.	1:5.186	1:5.216
		1½	75678 lbs. = 33.784 tons.	1:5.246	
Brymbo Iron, No. 3,	15508 lbs. = 6.923 tons.	¾	76133 lbs. = 33.988 tons.	1:4.909	1:4.936
		1½	76958 lbs. = 34.356 tons.	1:4.963	
Bowling Iron, No. 2,	13511 lbs. = 6.032 tons.	¾	76132 lbs. = 33.987 tons.	1:5.635	1:5.555
		1½	73984 lbs. = 33.028 tons.	1:5.476	
Ystalyfera Anthracite Iron, No. 2, . . .	14511 lbs. = 6.478 tons.	¾	99926 lbs. = 44.610 tons.	1:6.886	1:6.735
		1½	95559 lbs. = 42.660 tons.	1:6.585	
Yniscedwyn Anthracite Iron, No. 1, . . .	13952 lbs. = 6.228 tons.	¾	83509 lbs = 37.281 tons.	1:5.985	1:5.811
		1½	78659 lbs. = 35.115 tons.	1:5.638	
Yniscedwyn Anthracite Iron, No. 2, . . .	13348 lbs. = 5.959 tons.	¾	77124 lbs. = 34.430 tons.	1:5.778	1:5.712
		1½	75369 lbs. = 33.646 tons.	1:5 646	

109. The average of the sixteen irons in Table 107, which were taken from the Scotch, English, and Welch iron works, is 38.52 tons, and, including the cylinders and prisms in the three former Tables, § 103, &c., in one average, the resistance of cast iron to compression would be equal to 33.526 tons per square inch.

But this, probably, would be too low an estimate of its resistance, for, in the "Experimental Researches" published several years previous to the "Report" now quoted, which give the result of experiments by the same author, and carried on in the same manner, the average was 49.515 tons. From the mean of both we have 41.52: very nearly that given in the proposition, § 99.

110. Table 108 is of the utmost importance in the investigation of the proper form, in transverse section, of cast-iron beams and pillars. The ratio of the tensile and compressive resistances to be deduced from the propositions §§ 3 and 95, is 1 : 6, that is, 7 to 42. Mr. E. Hodgkinson's published results are—

	Tension. Tons.	Compression. Tons.
Experimental Researches on the properties of Cast Iron, ...	7.375	49.515
Report on Iron applied to Railway structures,	6.834	33.526

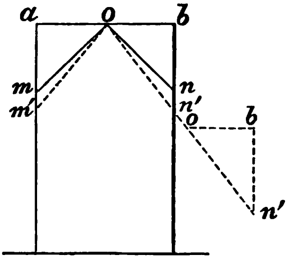
In both tension and compression the earlier experiments gave the higher results, the former 8, the latter 50 per cent. The same was remarked in § 30 of the tension of wrought iron.

111. A rigid body, like cast iron, which is of a crystalline and granular texture, begins to fail by suddenly splintering off in wedge-shaped fragments at the edges of the surface at which the pressure is applied: the compression, combined with the outward tendency of the particles, resulting in this method of failure, this substance not having sufficient ductility to admit of its failing by bulging laterally, which only takes place to a very small extent.

The fracture may take place either in one single oblique plane, or in several directed towards the different sides of the body; but in whatever mode, the plane invariably makes the same angle with the direction of the applied force, and this, in all the experiments recorded under this head, was in the vertical.

112. If we suppose the only forces acting on the wedge-shaped fragment to have been, the vertical force, and the cohesion, or rather shearing resistance, of the particles in the

oblique plane of fracture, we readily perceive that the angle it makes with the horizontal must be 45°. The wood-cut represents a prism, supposed to be compressed by a vertical force on the surface, *aob*. The point *o* is taken in the middle of *ab*. The resisting cohesive force on any section *om* or *on* varies as the lengths of these lines, that is, directly as the secant of the angle *aom* or *bon*, or inversely as the cosine. The compressive force acts in the vertical, and, consequently, its energy in forcing the portion *aom* along the inclined plane *om* is proportional to the sine of the same angle.



Now the body will fail with that inclination of *om* or *on* in which the ratio of the resistance on that surface, to the pressure, resolved in the direction of the inclined plane, is a minimum, that is, when $\frac{1}{\sin aom \times \cos aom}$ is a minimum, or when the denominator is a maximum. Now as

$$\sin aom \times \cos aom = \tfrac{1}{2}\sin 2aom,$$

the first expression is equal to $\frac{1}{\sin 2aom}$; and sin 2*aom* is a maximum when the angle *aom* is 45°.

113. The wood-cut shows the angle *aom* = 45°, and the dotted lines, *aom'* and *bon'*, the angle in which it is found by actual experiment that the splinter breaks off when the specimen fails; it is such that, in cast iron, *am'* is most generally equal $1\frac{1}{2}$ *ao*, making the angle *aom'* = 56° very nearly. Now, as there must be a force of friction, or analogous to friction, between the two inclined surfaces of fracture, and also those of the surface *ab*, and that of the steel bar by which the crushing pressure is applied (which last, as well as the weight of *aom'*, may be neglected), we may expect that angle to be greater than 45°.

114. The angle at which cast iron is fractured by a force of compression is determined by the following investigation:*

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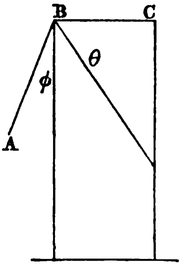
* This calculation is by Dr. Hart, S. F. T. C. D., late Professor of Mechanical Philosophy in the School of Engineering, Trinity College, Dublin. An investigation of a subject precisely analogous has been given independently by the Rev. S. Haughton, Professor of Geology in this University, in a Paper on "Reversed Faults," in the "Transactions of the Geological Society of Dublin," vol. viii., pp. 84, *et seq.*

Let W be the least weight which, being placed on the upper surface of the vertical pillar represented in the annexed wood-cut, will cause fracture in the oblique plane, having the angle θ with the horizon. Let



a . . . be the area of the transverse section of the pillar; then

$a \sec\theta$ will be the area of the oblique section made by the plane of fracture.

k . . . the coefficient of cohesion, i. e. a weight in tons per square inch, or other units.

f . . . the coefficient of friction, which is well known to be equal to the tangent of the angle of the plane of repose, which call $\tan\phi$.

Then the equation of equilibrium will be

$$W\sin\theta = ka\sec\theta + fW\cos\theta. \qquad (1)$$

Or, multiplying by $2\cos\theta$, and transposing,

$$W(\sin 2\theta - f\cos 2\theta - f) = 2ka\,; \qquad (2)$$

remembering that

$$2\sin\theta \times \cos\theta = \sin 2\theta, \text{ also } 2\cos^2\theta = \cos 2\theta + 1, \text{ and } \cos\theta\sec\theta = 1.$$

To find, then, the value of θ which will make W a minimum, it is only necessary to differentiate equation (2), and make $dW = 0$, which will give (since $d\sin 2\theta = 2d\theta\cos 2\theta$, and $d\cos 2\theta = -2d\theta\sin 2\theta$),

$$\cos 2\theta + f\sin 2\theta = 0\,; \qquad (3)$$

or,

$$\tan 2\theta = -\frac{1}{f} = \tan(90^\circ + \phi)\,;$$

hence

$$\tan 2\theta \,.\, \tan\phi + 1 = 0\,;$$

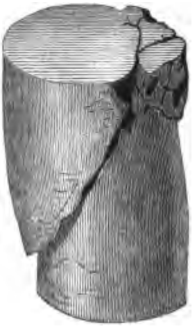
and also (Galbraith and Haughton, "Manual of Trigonometry," chap. v. (19)),

$$\tan 2\theta \,.\, \tan\phi + 1 = \frac{\tan 2\theta - \tan\phi}{\tan(2\theta - \phi)} = 0. \qquad (4)$$

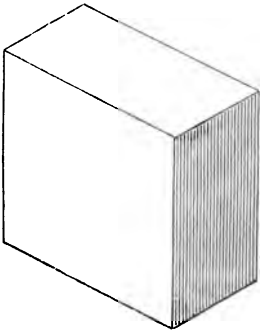
Hence the denominator must be infinite, or $2\theta - \phi = 90^\circ$, so that $2\theta = 90^\circ + \phi$, from which we derive a geometrical construction: lay off the angle of repose ϕ at the point B, and bisect the obtuse angle ABC; the line so drawn gives the plane of easiest fracture of the prism compressed on the surface BC. The angle θ being known to be 56°, we should have $\phi = 22°$. Thus, having in any material the value of either ϕ or θ, we can obtain the other.

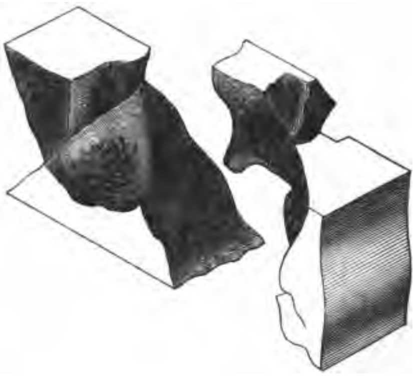
Experiments are now being undertaken to determine the angle of repose ϕ between two surfaces of cast iron, after being fractured.

115. The actual results of experiment showing the manner in which cast iron yields to a crushing force are delineated in the following wood-cuts, the first of which represents one of the cylinders, $\frac{3}{4}$ inch in diameter, and $1\frac{1}{2}$ inch in height, after it had been destroyed by compression. The single oblique plane, in which the sliding took place, is very clearly indicated,—a slight permanent bulging and bending having also occurred. This specimen has evidently yielded by the upper part slipping down by a sort of shearing action, the wedge having the whole upper end of the cylinder for its base, the angle being that found constant in cast iron, namely, the angle of the plane of least resistance.

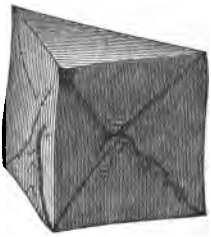


The two next engravings represent a whole and a fractured prism; in the latter, of which the sharp-pointed pyramid, with the lower side of the specimen for its base, is very

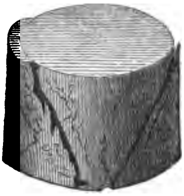




clearly shown; it has cut up the prism, separating the sides, and left a number of sharp-edged parts, all of which have slid off in the angle of least resistance.



The third shows a fractured prism, with an equilateral triangle for base, the sides of which were one inch in length, and the height also one inch; two pyramids formed as usual, with their bases at the ends, and the vertices toward the centre, have thrown out the angular part. The parts, so separated, have in prisms of every form a general resemblance, and the form of the pyramidal wedge in this, as in the former cases, has considerable interest, being that of least resistance in cast iron, and furnishes hints as to the best form of cutters for that metal. (*Vide* Holtzapffel's "Turning and Mechanical Manipulation," vol. ii., pp. 520, 538.)



116. The result of these experiments points out a minor limit to the length of the specimens intended for experiment, as the incipient bending did the major limit. They must not be less than the height, which will be sufficient to allow of a perfect separation of the plane of fracture. The wood-cut represents a cylinder of a height less than one and a half times its diameter, and the portions of two oblique planes of fracture at the usual angle for cast iron. The central part has its base resting upon the shelf that supported the cylinder during the experiment, and manifestly cannot slide off in the surface of easiest fracture, or else it yields by sliding off in some diagonal plane, not so inclined as the plane of least resistance, and thus the great increase of resistance to compression of the short specimens is completely explained.

117. The illustrations given in the preceding sections indicate that, although the angle which the plane of easiest fracture makes with the horizon be constant, yet the particular position which it assumes is not always the same. There may evidently be an infinite number of such planes around any point in the upper surface of the specimen, all inclined at the same angle of nearly 56°; and there is no reason, in the nature of the material itself, if it be perfectly homogeneous, why it should take one of these planes of section rather than another. In the greater number of cases, however, one plane

of fracture will be determined in preference to the rest, either by some want of uniformity in the distribution of the compressing force on the upper surface of the specimen, or by the material itself not possessing a homogeneous structure.

But if neither of these causes presents itself, we shall not have any particular plane determined. In that case, the fracture having no tendency to take place in one direction rather than another, will take place in all directions at once; and thus the surface of fracture will assume the surface of a double cone, of which the two component cones have a common apex, from which the sides of the column will break away.

118. The strength of columns to resist compression is thus constant for the same transverse area between the limits of 1½ diameters high, at the least, and from 3 to 5, at the greatest. For, every height of a column of constant diameter between these two limits, the section of easiest fracture being a plane inclined at the same angle to the axis of the column, must be a plane of constant area; so that in each case the cohesion of the same number of particles has to be overcome, that the fracture might be produced; and the cohesion of the same number of particles being to be overcome under the same circumstances for each different height, the same force would be required to overcome that cohesion, until, at length, the height was attained, at which the column would begin to bend. That height once reached, a pressure continually less as the column was longer becomes sufficient to break it, as is proved experimentally in Tables, §§ 124, 125. For the lesser heights below 1½ diameters the material gradually increases in strength, and at last, even if an unconnected granular mass, would resist without any limit or any law. In the shortest specimens that were crushed, speaking of those about ⅛ inch high, and ¼, ⅜, and ½ inch diameter, fracture takes place by the middle becoming flattened and increased in breadth, so as to burst the surrounding parts, and cause them to be crumbled and broken in pieces. The blank of a coin is a cylinder whose lateral dimensions greatly exceed the height; the milling on the edge is effected by the radial extension of the piece under the severe impact of the die, which would destroy it if of greater height, and unconfined at the sides. Between the heights of 3 or 5 diameters, and about 25 or 30, the column, though it may yield by slipping off in a diagonal plane, bends; above that it yields by transverse fracture from flexure, so that three distinct sets of laws govern the strength of columns, depending on the ratio of the height to the diame-

ter. The crushing resistance has been considered; of the other two, it will be convenient, first, to determine these laws as regards pillars whose height is not less than twenty-five times the diameter; and subsequently investigate the formula for the strength of those between twenty-five times and the height at which the resistance is constant.

119. The flexure thus observed leads directly to the consideration of a property of bodies no less important in practical construction than the ultimate resistance, namely, the ELASTICITY. For, every pillar or beam when bent, either by a force in the direction of the axis, as in a vertical pillar, or transverse to it, as in a beam lying horizontally between supports, is lengthened on the convex, and shortened on the concave side.

Mr. E. Clarke experimented on a bar of wrought iron, 16 inches long, and 1 inch square, laid upon supports 12 inches apart in the clear. With a weight of 2.33 tons suspended from the centre it deflected $2\frac{1}{4}$ inches; the top was shortened $\frac{10}{16}$ths of an inch; it must, therefore, have been under compression; the convex, or under side, was lengthened $\frac{13}{16}$ths of an inch; it must, therefore, have been in tension; and though a pillar, when bent by a compressing force, is not in an identical state with a beam bent by a transverse force, yet, in this case also, the convex must be longer than the concave side of the pillar, and resisting tensile and compressive forces respectively.

The form of the specimens used by experimenters in determining the ultimate direct tensile and compressive resistances of bodies was not suited to develop the elasticity, and, therefore, it has not been previously mentioned. It had been always studied indirectly, by measuring the deflections of horizontal bars, until Mr. E. Hodgkinson's great series of *direct* experiments, undertaken for the "Report" already quoted, who measured the extension of vertical rods, 50 ft. in length, and the compression of pillars 10 ft. high, restrained from bending; and also the degree in which, in each case, they returned to their first length, after each trial, when unloaded. These experiments will be given in a future page.

120. ELASTICITY, or Elastic Force, is that inherent property in bodies by which they recover their original figure after any force which has disturbed it is withdrawn. To account for this phenomenon, it has been supposed that the ultimate molecules, of which all bodies must be composed, have the power to attract and repel each other in such manner that, in the habitual or normal state, the forces of attraction and re-

pulsion, which act upon these molecules, are in equilibrium; and though of great intensity, have but a very small sphere of action, thus permitting of a very minute separation and approach of the ultimate particles. From this it would result, that every external tensile force tending to separate the particles, calls into play the force of attraction, and slightly elongates the body. And also every external force tending to compress them develops the force of repulsion, and contracts the length. When the external force ceases to act, the effect disappears also, and the body is restored to very nearly the same original figure and volume.

121. Solid bodies are thus capable of being extended under tensile, and shortened under compressive, forces. For example, a bar of cast iron 50 ft. long, and 1.058 square inch in transverse section, having been placed vertically, and a weight of 5 tons suspended from it, was observed to be 0.574 inch, or the $\frac{1}{1045}$th part greater in length than when unloaded, and on removing this tensile force, the bar was found again to assume very nearly its former unloaded dimensions, being now 50 ft. and 0.06425 inch long. The property by which it is thus restored to its former dimensions is its elasticity.

In like manner, a bar 10 ft. long, and 1.0816 sq. inch area, having been compressed by the same force of 5 tons, is observed to be shortened 0.09944 inch, or the $\frac{1}{1207}$th part; and again, on the removal of the pressure, it was found restored to very nearly its original length, being 0.00847 inch shorter than the original 10 ft.

In both tension and compression alike, a defect of elasticity was always observed, the bar never returning perfectly to its original length—a *set*, as it is called, being found to have taken place after the force had been removed.

122. It has been stated in § 102, and also after the Table, § 103, that the resistance of short columns to crushing was, *cæteris paribus*, proportional to the transverse area of the specimen. The peculiar manner in which the failure takes place in cast iron, as described in § 116, would lead to the presumption that it would be so found; for as the angle the diagonal section, in cast iron, makes with the direct transverse is constant, namely, the angle whose tangent is about $1\frac{1}{2}$ to 1, or 56°, they are in a constant ratio, that is, the oblique area is equal to the direct, multiplied by a constant quantity. And the strength being as the number of particles in the oblique plane of fracture, will be as the area of such plane, which is proportional to the direct transverse area.

Taking the average crushing weights for the several diameters out of Table § 103, we have—

Assuming that— The numbers in the column of the diameters are, in inches, . . .	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{4}$
Or as the numbers	1	2	3	4	5
And, therefore, the areas as . . .	1	4	9	16	25
The average crushing weight in lbs.	3194	13818	27232	46861	79421
Dividing these last, all by 3194, we have, ...	1	4.326	8.53	14.67	24.87

These numbers are nearly as the areas, and thus the strength of short prisms may be taken as proportional to the transverse area, that is, to the square of the diameter.

123. As the laws which determine the strength of long pillars are, in the following sections, deduced experimentally, it will be sufficient now, merely to state the deductions of mathematical writers on this subject, who have all followed the original theory of Euler. Having calculated the form of the curve assumed by a thin elastic lamina fixed at one end, or lying horizontally on two supports, they proceed to determine, in the case of a vertical elastic lamina, the *least* weight which, placed on the top, will suffice to produce flexure, and this was supposed to be the weight practically limiting the strength. The height and lateral dimensions being now supposed increased or diminished, it is proved that the strength is inversely as the square of the height,—on the supposition that the material is incompressible,—and as the fourth power of the lateral dimension, if the transverse sections are similar, or if rectangular, though not similar in transverse section, as the longer side into the cube of the other.

In the case of hollow pillars, the strength was shown to be proportional to the difference of the fourth powers of the diameters, or, in general terms, if h be the height of any pillar, and a, b its transverse dimensions, a being the greater, and d the diameter if circular, or any lateral dimension if the pillars are similar in transverse section: then the strength to resist incipient flexure, or the measure of the *least* weight sufficient to produce it, is $\frac{ab^3}{h^2}$, or $\frac{d^4}{h^2}$; and if d_1 be the internal diameter of a hollow column, it becomes $\frac{d^4 - d_1^4}{h^2}$.*

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* *Vide* Poisson's "Mecanique," vol. i., second edition; Venturoli's "Mechanics," translated by Cresswell, book iii., chaps. 17, 18, 19; Lloyd's "Mechanical Philosophy," section xi., Second Edition.

124. TABLE *showing the ultimate Resistance of solid uniform Cylinders of Cast Iron to a Force of Compression; the Ends of the Specimen being turned, and* rounded, *so that the Force would pass through the Axis.*

Low Moor iron, No. 3, cast in dry sand.

	Height.	Mean Diameter.	Weight of Pillar.	Mean Deflection at Middle Height.	Mean of Breaking Weight.	Value of x.*	
	Inches.	Inches.	lbs. oz.	Inches.	lbs.		
I.	60.5	0.50	. . .	0.36	143	30309	Mean value of x, 33379 lbs. = 14.9 tons.
II.		0.77	. . .	. .	780	32603	
III.		0.99	. . .	0.52	1902	30902	
IV.		1.29	19 11	0.38	5707	34273	
V.		1.527	28 7	0.14	10755	34229	
VI.		1.77	38 0	0.33	16562	30092	
VII.		1.96	46 0	0.54	23551	29901	
VIII.	30.25	0.50	. . .	0.12	539	35162	
IX.		0.77	. . .	0.10	2726	35070	
X.		0.99	. . .	0.10	6105	30529	
XI.		1.29	. . .	0.14	17235	31857	
XII.		1.52	. . .	0.10	32531	32447	
XIII.	20.166						
XIV.		0.767	2 7	. .	6602		
XV.		1.01	4 4	. .	15737	36639	
XVI.	15.125	0.50	. . .	0.28	1904	38230	
XVII.		0.76	. . .	. .	9223	38360	
XVIII.		0.99	. . .	. .	19752	30401	
XIX.	12.1						
XX.							
XXI.	10.0833	0.76	. . .	. .	17506	36547	
XXII.							
XXIII.	7.5625	0.497	. . .	. .	5262	33263	
XXIV.		0.777	. . .	. .	22948		
XXV.	3.7812	0.50	. . .	. .	15107		
XXVI.							
XXVII.							

* This column is explained in §§ 127, 133.

125. TABLE *of the Results with solid uniform Cylinders, but with Ends turned* flat *and parallel, being compressed between two parallel Surfaces coinciding with Ends of Cylinder, as in* § 103, *&c.*

Low Moor Iron, No. 3, cast in greensand.

	Height.	Mean Diameter.	Weight of Pillar.	Mean Deflection at Middle Height.	Mean of Breaking Weight.	Value of *x*.	
	Inches.	Inches.	lbs. oz.	Inches.	lbs.		
I.	60.5	0.51	3 $4\frac{3}{4}$	. .	487*	83182*	
II.		0.77	7 $5\frac{1}{2}$	0.24	2456	97175	Mean value of *x*, 98922 lbs. = 44.16 tons.
III.		0.997	11 8	0.18	6238	98638	
IV.		1.29	19 12	0.46	16064	101770	
V.		1.56	29 0	0.55	28962	93455	
VI.							
VII.							
VIII.	30.25	0.50	1 $10\frac{1}{2}$	0.13	1662	93735	
IX.		0.77	. . .	0.21	8811	107300	
X.		1.01	. . .	0.16	20310	94403	
XI.							
XII.							
XIII.	20.166	0.51	1 $1\frac{3}{8}$	. .	3830	101063	
XIV.		0.777	2 $8\frac{1}{3}$	. .	15581	92229	
XV.		1.022	4 $5\frac{1}{2}$	. .	31804		
XVI.	15.125	0.51	0 $13\frac{10}{12}$	. .	6764	109445	
XVII.		0.775	0 $29\frac{3}{16}$	. .	21509		
XVIII.		1.00	3 4	. .	40250		
XIX.	12.1	0.50	0 $10\frac{1}{8}$	. .	7195		
XX.		0.785	1 $7\frac{3}{4}$	. .	24287		
XXI.	10.0833	0.50	0 $8\frac{1}{2}$	. .	8931		
XXII.		0.768	. . .	. .	25923		
XXIII.	7.5625	0.50	0 $6\frac{1}{2}$	. .	11255		
XXIV.		0.777	0 $14\frac{5}{12}$	. .	32007		
XXV.	3.7812	0.50	. . .	. .	17468		
XXVI.	2	0.52	. . .	. .	22867		
XXVII.	1	0.52	. . .	. .	24616		

The two pillars from which this average was taken had disks two inches in diameter cast on their ends: all the others were turned flat and parallel. From the difficulty of obtaining slender pillars, quite straight, this number was omitted in the mean.

126. In the experiments tabulated above, the object has been to determine, not the *least* weight producing incipient flexure, but that producing fracture by flexure, as leading to a practical rule for the due proportioning of columns to the duty they have to perform in actual construction. In deducing these rules, Mr. Hodgkinson acknowledges that some of the theoretical results given by Poisson have been of great service in the course of his inquiry. Though he also writes:—"I have many times sought, experimentally, with great care for the weight producing incipient flexure, according to the theory of Euler, but have hitherto been unsuccessful. So far as I can see, flexure commences with weights far below those with which pillars are usually loaded in practice. It seems to be produced by weights much smaller than are sufficient to render it capable of being measured. I am, therefore, doubtful whether such a fixed point will ever be obtained, if, indeed, it exist. With respect to the conclusions of some writers, that flexure does not take place with less than about half the breaking weight, this, I conceive, could only mean large and palpable flexure; and it is not improbable that the writers were in some degree deceived, from their having generally used specimens thicker, compared with their length, than have been usually employed in the present effort."

Mr. E. Hodgkinson's valuable series of experiments on uniform pillars of cast iron to resist fracture by a crushing force are contained in four Tables which have been condensed in this work. The first on *solid* uniform pillars, rounded at the ends; and the second, the same as the first, but with ends turned flat and parallel, have been given above. The third, on *hollow* uniform pillars, with a hemispherical cap fitted on the ends; and the fourth, the same as the third, but turned flat and parallel at the ends, are given in a future page.

The experiments enumerated in the above Tables have been skilfully arranged, so as to point out very clearly the law of the strength of pillars as depending on the dimensions. The shorter lengths are exact submultiples of the length of the longest pillar; this was intended to have been 60 inches, but, by an oversight, it was made 60.5 inches instead; and of this last, the others were cast respectively the half, the third, the fourth, fifth, sixth, eighth, and sixteenth parts. The diameters are from ½ an inch to 2 inches, increasing by ¼ inch with nearly every different height. Now, in order to determine the law as dependent on length, we must take out from the Table those results in which the diameter is the same, but the length different, as, for instance, lines I., VIII., XIII., XVI., XIX.

Or again, taking out lines II. IX. XIV., &c., in each of which the constant diameter is 0.77 in., but the lengths different, and in some simple ratio to each other. Then, from a comparison of the ratio of the lengths with that of the breaking weights, we can determine the law which connects the results of the experiments in this respect.

It will be at once perceived, from both Tables, that pillars are, *cœteris paribus*, stronger as the length is diminished, the greater breaking weights being opposite the shorter lengths, and therefore it must be some *inverse* proportion which obtains between the quantities compared together.

Comparing then the lengths and breaking weights of solid pillars, with ends turned *flat*, in lines II. and IX., § 125, page 87, we may calculate the power of the length to which the breaking weights are inversely proportional. The constant diameter, in these two lines above mentioned, we find to be 0.77 in., the lengths being 60.5 and 30.25 in., and the breaking weights 2456 lbs. and 8811 lbs. respectively.

Let n be the unknown power to which these lengths must be raised, to make them in the inverse ratio of the breaking weights. We have

$$60.5^n : 30.25^n :: 8811 : 2456,$$

or

$$2^n : 1^n :: 8811 : 2456.$$

It is evident from this proportion that n is somewhat less than 2, since the third term is not equal to 4 × 2456, and the power of the length, to which the strength of pillars of the same diameter is proportional, is less than the inverse square.

To determine its exact value, we have, from the above proportion,

$$2^n = 8811 \div 2456 = 3.58754,$$

or

$$n \times \log 2 = \log(8811 \div 2456),$$

and

$$n = \frac{\log(8811 \div 2456)}{\log 2} = \frac{0.5547919}{0.3010300} = 1.843;$$

hence

$$60.5^{1.843} : 30.25^{1.843} :: 8811 : 2456.$$

In like manner, from all suitable experiments, we obtain values of n, the average of which will be a safe guide in calculating the ultimate strength of pillars used in practical construction.

The Tables given in pages 86 and 87, have however been since very greatly extended by the same experimenter, namely, up to 10 ft. in length, and $2\frac{1}{2}$ in. diameter. As it will be necessary to include the results of these experiments, in determining the average index of the lengths, the following condensed summary is now introduced. In the Table will also be found a column giving the crushing weight per square inch of the various irons, and the same for an area equal to 2½ in. diameter, or 4.909 square inches.

TABLE *supplemental to that in* § 125, p. 87, *showing the Breaking Weights* (B.) *of Long Solid Pillars of Cast Iron, from various parts of the kingdom,* 2½ *inches in Diameter, and the Crushing Weight* (C.) *of short specimens of the same.*

No.	Names of the Iron Works.	SHORT PILLARS.		LONG PILLARS.				Value of the constant x from 10 ft. pillars.
		Crushing weight in tons.		2½ in. diameter. Breaking weight in tons.				
		Per sq. in. of section.	For 2½ in. diameter.	Length, 10 ft.	Length, 7½ ft.	Length, 6¼ ft.	Length, 5 ft.	
1	Old Park, . .	39.3	195.3	29.50	44.73	. .	. .	49.94
2	Derwent, . .	43.4	216.9	28.03	47.04	50.20	. .	46.91
3	Portland, . .	42.3	209.4	27.30	45.78	. .	. .	46.47
4	Calder, . . .	37.4	186.2	27.09	41.58	. .	. .	46.49
5	London(*a*), . .	36.1	183.1	25.41	41.79	51.04	. .	41.46
6	Level, . . .	30.5	150.6	24.67	43.89	58.60	. .	42.05
7	Coltness, . . .	28.1	138.3	23.52	38.85	. .	. .	40.23
8	Carron, . . .	30.6	151.1	23.52	32.34	. .	. .	40.16
9	Blaen Avon, .	31.7	154.5	22.05	32.13	. .	52.51	38.44
10	Old Hill, . .	24.5	122.1	20.05	. .	41.79	57.69	33.60
11	London(*b*), . .	42.6	209.5	27.19	. .	54.49	. .	46.21
12	Low Moor, . .	34.6	176.5	24.99	39.90	. .	. .	40.48
13	Blaen Avon, .	37.3	190.6	25.62	38.22	. .	. .	41.22
			C.	B.	B.	B.	B.	
	I.	II.	III.	IV.	V.	VI.	VII.	VIII.

The quality of the iron was chiefly No. 1. That from Low Moor, in line 12, was No. 2; that from Blaen Avon, line 13, was No. 3. The London(*a*), line 5, was a mixture, one-half old plate-iron, and one-half Calder iron. London(*b*), line 11, was two-thirds old metal, and one-third of No. 1 best Scotch pig-iron. The quality experimented upon in the Tables in pages 86, 87, was exclusively No. 3, and from Low Moor only. The iron in lines 2, 3, 4, 6, 7, 8, was hot blast; the others cold blast.

The pillars were all cast erect, and in dry sand, from the same iron model, nearly 12 ft. long, to insure sound castings; they were afterwards cut to 10 ft. long, and their ends turned flat and perpendicular to the axis. Two pillars were cast from each kind of iron; the tabulated breaking weight is thus in each case an average of two experiments. The weights of the pillars, down to line 11 inclusive, were, on an average, 152 lbs. each; the average of line 12 was 159½, and of line 13, 160 lbs.

The Old Park Iron Works are situated at Stourbridge, in Staffordshire; the Derwent, in Durham; the Portland, at Tovine, in Scotland; the Calder, in Lanarkshire; the Level Iron Works, in Staffordshire; the Coltness, in Edinburgh; the Carron, in the county of Stirling; the Blaen Avon, in South Wales; the Old Hill, in Staffordshire; the Low Moor, in Yorkshire.

Calculating exactly as in the example, page 89, we have the following Table of values of the power of the length *n*, to which the breaking weights are proportional:—

TABLE *showing the Law of the Strength of Long Solid Pillars of Cast Iron as dependent on their Length, that is, the Power to which the Lengths are raised to give them the inverse ratio of the Breaking Weights. Ends flat and perpendicular to the axis.*

Diameters in inches.	Lengths compared in inches.	Inverse breaking weights in tons.	Value of index *n*.
2½	120 and 90	Average of 10 expts.	1.6303
"	120 " 75	58.60 and 24.67	1.8864
"	120 " 75	51.04 " 25.41	1.5737
"	120 " 75	41.79 " 20.05	1.5514
"	120 " 75	50.20 " 28.03	1.4137
"	120 " 75	56.89 " 28.35	1.4820
"	120 " 75	52.09 " 26.04	1.4752
1½	120 " 75	11.055 " 5.00	1.8338
"	120 " 75	11.57 " 4.85	1.8986
2½	120 " 60	60.70 " 19.95	1.4523
"	120 " 60	55.01 " 19.95	1.5942
1	60.5 " 20.1666	14.2 " 2.78	1.4830
"	60.5 " 30.25	9.07 " 2.78	1.6910
0.77	60.5 " 15.125	9.6 " 1.096	1.5500
"	60.5 " 20.1666	6.956 " 1.096	1.6820
"	60.5 " 30.25	3.93 " 1.096	1.8430
General mean value of *n* from 25 comparisons, .			1.6275

So that we may assume

$$n = 1.63.$$

"In a former paper," remarks Mr. E. Hodgkinson, "I assumed $n = 1.7$ as the inverse power, which the strength of pillars of the same diameter, and of different lengths, followed nearly; this applying both to pillars with rounded ends, and to those with flat ends; but it appears, from above, that in flat-ended pillars, which represent the great mass of those used in practice, the strength would be better represented by some number intermediate between 1.6 and 1.7: the results of the experiments here detailed give 1.63 as a mean nearly, and this number may be used for all pillars whose length is more than about thirty times the diameter."

Thus two pillars, one of which is 10 ft. long, and 1 in. in diameter; the other, 1 ft. long, and also 1 in. in diameter, have their breaking weights in the proportion $\frac{1}{10^{1.63}}$ to $\frac{1}{1^{1.63}}$, or $\frac{1}{42.658} : 1$, that is, $0.023442 : 1$, or $2.3442 : 100$.

If the lengths had been 10 ft. and 5 ft., the diameters being the same, the breaking weights would have been in the ratio $\frac{1}{10^{1.63}} : \frac{1}{5^{1.63}}$, or $\frac{1}{42.658} : \frac{1}{13.7823}$, or $0.02344 : 0.0726$, that is, 1 to 3 nearly.

If the lengths be taken at 10 ft. and 6½ ft., the diameters being equal, the breaking weights are in the ratio $\frac{1}{42.658} : \frac{1}{21.137}$, which is very nearly 1 : 2. Thus at about two-thirds of the length the strength of pillars is doubled, and at half the length nearly trebled.

By a method precisely analogous is determined the law of the strength of pillars as depending on their diameter, or lateral dimension, when not circular; taking out in this case those which have the same length, but different diameter; as in the former we compared those which had the same diameter, but different length. We can then calculate the power m to which the two different diameters must be raised, in order to make them have the same ratio to each other as their respective breaking weights. That the proportion is *direct* will be perceived as readily in this case, as that it was *inverse* appeared in the former.

Thus, in long pillars with their ends flat, we have from Table, § 125, p. 87, a number of experiments, lines II. to V., in which the length is constant, being 60.5 in., but having four different diameters. Taking out from lines II. and V. the diameters and breaking weights, we have

$$0.77^m : 1.56^m :: 2456 : 28962.$$

As the diameters are in the ratio of 1 to 2, very nearly, it is easy to see that the value of *m* is between 3 and 4; the fourth term being more than nine and less than sixteen times 2456. The above proportion may be thus expressed by dividing

$$1^m : 2.026^m :: 1 : (28962 \div 2456 =)\ 11.7923,$$

and therefore, as from this

$$2.026^m = 11.7923,$$

we have $$m \times \log 2.026 = \log 11.7923,$$

and $$m = \frac{\log 11.7923}{\log 2.026} = \frac{1.0715875}{0.3066394} = 3.495,$$

so that $$0.77^{3.495} : 1.56^{3.495} :: 2456 : 28962.$$

By this method of calculation we obtain the following values of *m*, from lines II. and V. inclusive :—

TABLE *showing the Law of the Strength of Long Solid Pillars of Cast Iron as dependent on their Diameter. Ends flat and perpendicular to axis.*

No.	Lines compared.	Diameters compared in inches.	Breaking weights compared in tons.	Value of *m*.
	Lines.			
1	II. and III.	0.77 and 0.997	1.0964 and 2.7848	3.606
2	II. ,, IV.	0.77 ,, 1.29	1.0964 ,, 7.1714	3.641
3	II. ,, V.	0.77 ,, 1.56	1.0964 ,, 12.9295	3.495
4	III. ,, IV.	0.997 ,, 1.29	2.7848 ,, 7.1714	3.670
5	III. ,, V.	0.997 ,, 1.56	2.7848 ,, 12.9295	3.428
		Mean, ...		3.568

From a set of experiments on pillars 60.5 in. long, with circular discs on their ends, about double the diameter of the pillar, to prevent them turning, given in page 438, "Phil. Trans.," part ii., 1840, we have—

No.	Diameters compared in inches.	Breaking weights compared, in tons.	Value of *m*.
6	0.51 and 1.00	0.2174 and 3 0491	3.922
7	0.51 „ 1.28	0.2174 „ 7.3076	3.820
8	0.51 „ 1.53	0.2174 „ 13.7451	3.775
9	0.775 „ 1.53	1.2138 „ 13.7451	3.568
10	0.775 „ 1.28	1.2138 „ 7.3076	3.578
11	1.00 „ 1.53	3.0491 „ 13.7451	3.412
		Mean, ...	3.679

Each of these diameters and breaking weights are the average of at least two pillars. As in determining the general mean value of *n*, the inverse power of the length, Mr. Hodgkinson's later and larger experiments were introduced, so in determining the power of the diameter *m*, it is advisable to enlarge the basis from which so important a number is derived.

The following comparisons are from pillars 120 in. in length, ends turned flat, and of the quality named London Mixture:—

No.	Diameters compared in inches.	Breaking weights, or those under which the pillar was evidently sinking, in tons.	Value of *m*.
12	1.530 and 2.511	5.0 and 28.348	3.5016
13	1.541 „ 2.511	4.852 „ 28.348	3.6154
14	1.530 „ 2.496	5.0 „ 26.038	3.3708
15	1.541 „ 2.496	4.852 „ 26.038	3.4840
		Mean, ...	3.4929

The same pillars—not having been actually fractured—were rendered quite straight, and cut to be 75 in. long, the ends being turned flat.

No.	Diameters compared in inches.	Breaking weights compared, in tons.	Value of m.
16	1.530 and 2.511	11.055 and 56.89	3.3068
17	1.541 „ 2.511	11.58 „ 56.89	3.2603
18	1.530 „ 2.496	11.055 „ 52.086	3.1670
19	1.541 „ 2.496	11.58 „ 52.086	3.1179
	Mean, ...		3.2130

The general mean derived from these four means, which represent nineteen comparisons, gives 3.513, so that we may assume

$$m = 3.5.$$

127. Having thus determined the law, both as to length and lateral dimensions, by which the breaking weight of long solid pillars of cast iron, flat at the ends, and in length not less than thirty times the diameter, are governed, namely, that

$$\text{Breaking weight} \propto \frac{d^{3.5}}{l^{1.63}},$$

it remains to calculate a constant which we may call x, so that

$$\text{Breaking weight} = x \times \frac{d^{3.5}}{l^{1.63}}.$$

In practice, cast iron columns are, as to their dimensions, generally spoken of as being so many feet in length, and so many inches in diameter, and therefore a convenient constant will be the calculated breaking weight of a column 1 ft. long, and 1 in. diameter: we may easily determine this, by two successive operations, from any of the results of the experimental pillars. First, If W be the known breaking weight of any experimental pillar, whose length is l feet, and diameter d inches; then the breaking weight W′, of another pillar of the same length, and 1 in. diameter, is found by the proportion,

$$W : W' :: d^{3.5} : 1^{3.5}, \text{ hence } W' = \frac{W}{d^{3.5}},$$

the breaking weight of a column l feet long, and 1 in. diameter. Secondly, From W′, so determined, and the length l, we may next calculate the breaking weight x of a unit pillar 1 ft. long, and 1 in. diameter, by the inverse proportion

$$W' : x :: 1 : l^{1.63}, \text{ hence } x = W' \times l^{1.63} = W \times \frac{l^{1.63}}{d^{3.5}}.$$

Taking a particular case, as suppose a pillar 10 ft. long, and 2½ in. diameter. First calculate, from the known breaking weight of this, the breaking weight of a pillar of the same length, but diameter 1 in., merely reducing it in the direct proportion of $2.5^{3.5} : 1^{3.5}$, or 24·705 : 1—since 3·5 × log 2·5 answers to the former number—equivalent to $\frac{W}{d^{3.5}}$ giving W′. Secondly, we must increase this so found weight in the inverse ratio of the lengths raised to the 1.63 power, or $\frac{1}{10^{1.63}} : \frac{1}{1^{1.63}}$, that is $\frac{1}{42.658} : 1$, or 0.023442 : 1, equivalent to $W'l^{1.63}$.

Thus, one of the two pillars of Blaen Avon iron No. 3,—the average breaking weight of which is given at the intersection of lines 13 and IV., Table, p. 90,—had a breaking weight W of 25.41 tons; from this, which was 10 ft. long, and 2.556 in. diameter, we obtain the strength of another, also 10 ft. long, but only 1 in. diameter, by the proportion

$$26.697 : 1 :: \overset{\text{Tons.}}{25.41} : \overset{\text{Tons.}}{0.9518} = \frac{W}{d^{3.5}} = W',$$

as 3.5 × log 2.556 = 1.4264628 answers to 26.697.

Again, the strength of this pillar 10 ft. long, and 1 in. diameter, is to that of another 1 ft. long, and same diameter, in the inverse proportion

$$0.023442 : 1 :: \overset{\text{Tons.}}{0.9518} : \overset{\text{Tons.}}{40.6023} = x = W\frac{l^{1.63}}{d^{3.5}}.$$

The other had a breaking weight W of 25.83 tons, and diameter 2.546 in., hence

$$(3.5 \times \log 2.546 = \text{number})\ 26.333 : 1 :: 25.83 : 0.981 = \frac{W}{d^{3.5}},$$

and again, $0.023442 : 1 :: 0.981 : 41.85 \text{ tons} = x = W\frac{l^{1.63}}{d^{3.5}}.$

The mean of these two pillars of Blaen Avon, No. 3, being 41.22, as given at the intersection of lines 13 and VIII. By taking an average from a sufficiently extended series of experiments we obtain a safe value of this constant for practical purposes of construction. Thus, the mean of the numerous irons from all parts of the kingdom, given in column VIII. of the supplemental Table, page 90, is 42.59. Mr. Hodgkinson (Phil. Trans., 1857) adopts 42.347, which we may assume to be 42.35, being an average, derived from eight experiments on hollow pillars, from 2½ to nearly 4 inches external diameter, and two on solid pillars, 2¼ inches diameter (given in lines 15 to 24, Table § 135), all 10 feet long, and of Low Moor iron, No. 2, the ends being turned flat and perpendicular to the axis. The former experimental researches (Phil. Trans., 1840) of this author had given the higher number 44.16 for solid pillars, with flat ends, which is that given in column "Value of *x*," Table § 125, for Low Moor iron, No. 3. Although the Low Moor, No. 2, is stronger than No. 3, in the ratio of 1026 to 1000, yet the constant is, as we have seen, lower in the later experiments on No. 2, the explanation being that cast-iron, in larger masses, is always softer than that in smaller castings. And, as in actual construction the pillars are larger, in general, than the largest experimental pillars, we must take this smaller constant in preference to 44.16, given in page 87.

128. Having thus ascertained the inverse power of the length *n*, the power of the diameter *m*, and the constant 42.35 tons, being the breaking weight of a flat-ended solid pillar, 1 ft. long, and 1 in. in diameter, we can calculate the breaking weight of a pillar of any given dimensions, as suppose 13 ft. in height and 3¼ in. diameter.

$$\text{Breaking weight} = 42.35 \times \frac{3.25^{3.5}}{13^{1.63}} = 42.35\,\frac{61.89}{65.423} = 40.06 \text{ tons.}$$

And if the pillar were 9 ft. high, and 3¼ in. in diameter.

$$\text{Breaking weight} = 42.35 \times \frac{3.25^{3.5}}{9^{1.63}} = 42.35\,\frac{61.89}{35.93} = 70.6 \text{ tons.}$$

If in each of these two different heights of pillar, we suppose the diameter increased by half an inch, so that it becomes 3.75 inches, which, raised to the 3.5 power, is equal 102.12; then the breaking weights are

$$42.35 \times \frac{102.12}{65.423} = 66.11 \text{ tons, and } 42.35 \times \frac{102.12}{35.93} = 120.27$$

tons respectively: the areas of transverse section in each case being 9.085 sq. inches, and 11.05 sq. inches, or in the ratio of

1 to 1.216, in which ratio the quantities of metal are increased, but the breaking weights are in the ratio 1 to 1.7 nearly.

Before proceeding to the subject of hollow pillars, by which the advantages of cast-iron in construction are much more fully developed than by the use of solid pillars, it will be more convenient now to note the other interesting deductions from the experiments tabulated above, pp. 86, 87, 90.

129. The strength of long pillars with flat ends, turned true, perpendicular to the axis, and well-bedded, is about three times greater than that of pillars with the ends rounded, the diameter and length being the same.

It would appear, at the first view, that pillars with hemispherical ends, placed truly vertical, under a pressure also acting in the vertical, would sustain as much as those of the same transverse area and length having flat ends and a true bearing, and being under a pressure identical in amount and direction; and so perhaps it would prove to be, but for the different manner of their deflection, and consequent different fractures,—the round-ended pillar having one single curvature commencing at each end, like the deflection of a horizontal beam lying loose upon its supports, and having, like it, only one fracture at, or very near, the middle point. The flat-ended pillar, on the other hand, has three distinct curvatures, and three fractures, at the centre, and at each end, being like a horizontal beam fixed firmly on its abutments.

The proposition enunciated above may be proved by comparing the mean values of the columns in § 124 and 125, headed "Value of *x*," that for round-ended pillars being 14.9 tons, and for flat-ended 44.16; the latter being nearly three times greater than the former.

Now, although these values were deduced from the experiments, by using fractional indices for the diameter and length, different from those determined from the later and larger experiments of Mr. Hodgkinson (the average indices from the earlier experiments having given the law of the strength as $\frac{d^{3.55}}{l^{1.7}}$, instead of $\frac{d^{3.5}}{l^{1.63}}$, as now finally settled from the larger pillars), yet the application of the law $\frac{d^{3.5}}{l^{1.63}}$ will also give numbers whose mean value will have the ratio 1 to 3.

If we add together the mean breaking weights given in lines I., II., III., IV., V., and VIII., IX., X., and XVI. in Table § 124, page 86; and the same lines in Table § 125, we find the sums to be 30,561 lbs. and 91,754 lbs. respectively, which numbers are obviously very nearly as 1 to 3; the smaller being the sum of the breaking weights of pillars with round, the larger that of those with flat ends, which is only 71 lbs. greater than three times the former.

The principle followed in the selection of lines from the Tables made above, is to include those only whose length was not less than thirty times the diameter. In pillars shorter than this proportion, the strength of those with round to that of those with flat ends is not in a constant ratio; and it is found that it decreases when the length compared with the diameter decreases.

This relation between pillars with round and with flat ends is of great practical interest: it disposes of the recommendation of Serlio (and of Tredgold after him), to round the ends of struts of timber, giving them circular abutting joints, which we see would be voluntarily throwing away two-thirds of the full strength of the material. It is true that from settlements, and other causes, the compressing force is liable, with flat-ended struts, to take a direction in the diagonal; and from experiments, not yet mentioned, this would entail a loss of strength of about two-thirds. Notwithstanding, as this derangement or partial change in the position of the strained pieces may never occur, and may often be avoided, it cannot be prudent to give permanence and certainty to the contingency feared, by adopting at the first rounded ends.

It also shows, conclusively, the value of a true and flat bearing in all parts of structures under a compressive force, and that the ends should be firmly fixed. Connecting rods of steam-engines, and all rods of parallel motions, and moving machinery exposed to compression, are instances in practice of pillars with rounded ends, for their circular bearings are equivalent to rounded ends,—they bending or turning on the joint-pin as upon rounded ends.

Since, then, the strength of pillars with both ends rounded, and both flat, is found to be in a constant ratio, it seems probable, therefore, that the strength—when one end is fixed, and the other rounded—must be related to the results of the other cases by some very simple law. A series of experiments to test this truth, gave results as in the following abstract. The pillars specially used for this investigation were, for the most part, 30¼ inches long, and either 0.78 or 1 inch diameter:—

TABLE *showing the relative strength of the three kinds of Pillars, as described below; the Breaking Weights in lbs.*

		lbs.	lbs.	lbs.	lbs.	lbs.
Both ends rounded, ...		143	3017	7009	7009	16493
One ended rounded and one flat,		256	6278	13499	13565	33557
Both ends flat, or with discs, .		487	9007	20310	22475	
In inches.	Diameters, . . .	0.50	0.78	1.00.		1.75
	Lengths, ...	60.5	30.25			60.5

The number 3017 is the mean of three, and 7009 of five experiments.

The pillars in each vertical column in this abstract are of the same length and diameter. The strengths are obviously very nearly as 1, 2, 3 in the three different kinds of pillars,—the middle being an arithmetic mean between the other two. Other experiments on wrought-iron, timber, and steel, have shown that this law obtains in these materials also.

We have already seen (p. 92) that a pillar one-half the length of another, of the same diameter, each having flat ends, has three times the strength of the longer. And as a pillar with rounded ends has but one-third the strength of one with flat ends, it is evident that a long pillar with flat ends is of equal strength with one of half the length, and having round ends; the diameters being the same in each comparison,—the gain of strength from halving the length being compensated by the loss from rounding the ends.

The case of the ultimate strength of pillars fixed at one end, and movable at the other, is not without its interest in practice: as piston rods of steam-engines and the rods of force-pumps are instances of the use of materials in that form,—it being the same thing in effect whether the pillar be rounded at the end, or turn upon a joint there. Other instances may often be seen in practice; as where pillars are firmly based at the bottom, but through imperfect fixings, or some other cause, are capable of moving at the top.

The cause of the different strength of pillars, according to the form of the ends, as shown in this section, may in part be referred to their different mode of fracture.

The pillar with rounded ends breaks at or very near the centre, and with one fracture.

The pillar with both ends flat, and parallel to the axis, breaks generally at three places—in the centre, and near each end.

The pillar with one end flat, and the other round, breaks with one fracture, but at a point distant from the rounded end one-third of the length.

A very strong analogy, as has been already mentioned, may be traced in the three cases, and that of beams laid horizontally,—having either both ends lying free on the abutments, or both fixed; and, thirdly, one end free, and the other built in or fixed on the abutment.

The Plate opposite (page 102) represents each different form of pillar under the action of a vertical compressing force, and supposed deflected up to the point of fracture,—each having on one side a curve indicating the fracture.

The following is Mr. Eaton Hodgkinson's suggested explanation of the different resisting powers of long pillars, under the various circumstances of length, and form of termination:—

"Suppose a long uniform bar, or bolt of cast iron, were bent by a pressure at its ends, so as to take the form A*bcdef*B (Fig. 4 on Plate opposite, page 102); where all the curves A*bc*, *cde*, *ef*B, separated by the straight line A*ce*B, would be equal, since the bar was supposed to be uniform. The curve having taken this form, suppose the points *b* and *f* to be rendered immovable by some firm fixings at these points. This done, it is evident we may remove the parts near to A and B, without at all altering the curve *bcdef* of the part of the pillar between *b* and *f*, and consider only that part. The part *bf*, which alone we shall have to consider, will be equally bent at all the points *b*, *d*, *f*. The parts *c* and *e*, too, are points of contrary flexure; consequently, the pillar is not bent in them. These points are unconstrained, except by the pressure which forces them together; and the pillar might be reduced to any degree in them, provided they were not crushed or detruded by the compressing force. These points may then be conceived as acting like the rounded ends of the pillars in the Table § 124, page 86, and the part *cde* of the pillar, with its ends *c* and *e* supposed to be rounded, will be bearing the same weight as the whole pillar *bcdef*, of double the length, with its ends *bf* firmly fixed.

That the strain at all the points *b*, *d*, *f* is nearly equal, was shown by the experiments in the supplemental Table, page 90, upon pillars varying in length from 6 ft. 3 in. to 5 ft., and 2 ft. 6 in.; the fracture in some cases taking place at all three points at the same time, and in others at two of them,—showing the fracture to have commenced on opposite sides of the middle and of the ends, as might be inferred from Fig. 4, opposite page 102. The experiments on the 2 ft. 6 in. pillar are not tabulated in page 90; but in "Phil. Trans." for 1857, p. 886, it is stated that they broke in three places—in the middle, and at each end.

130. Similar pillars of cast iron have their strengths, or breaking weights, very nearly as the square of the linear dimensions,—the power being about 1.865 on an average.

The law of the strength of similar pillars may be deduced from the results of the experiments in the same Tables, § 124 and 125, which, from their excellent arrangement, and from the design of the experiments themselves, have been already found so fertile in practical consequences.

Pillars are similar when their diameters are to each other as their lengths: thus the pillars in lines V. and XIII., § 125, are similar,—the diameters being 0.51 and 1.56 inches, and the lengths 20.166 and 60.5 inches, respectively,—the ratio being evidently

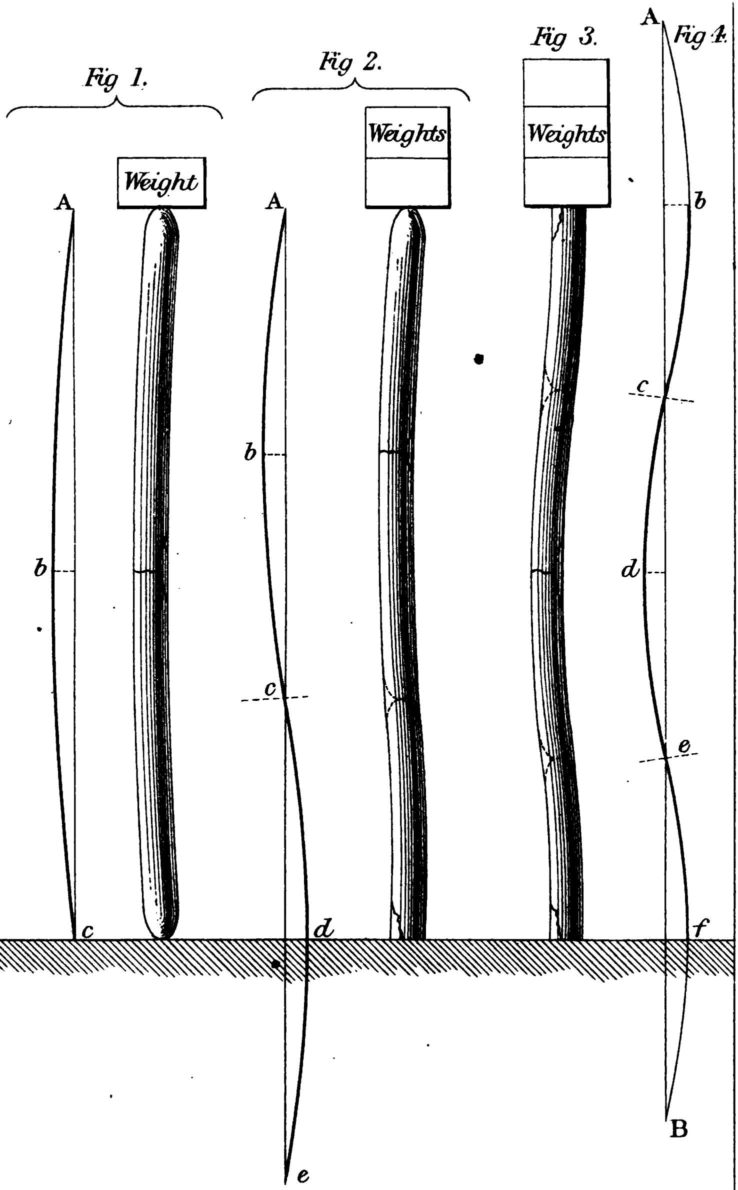
$$0.51 : 1.56 :: 20.166 : 60.5 :: 1 : 3.$$

That is, the one pillar is to the other, in lineal dimension, as 1 to 3: their weights will therefore be as 1^3 to 3^3, or 1 to 27; and examining the column headed "Weight of Pillar," we find that they are, respectively, 1 lb. 1⅝ oz. and 29 lbs.,—the latter being very little less than twenty-seven times the former. The breaking weights are 3830 lbs. and 28,962 lbs., being in the ratio of 1 to 7.562; these numbers being evidently a little lower than in the ratio of the square of 1 to 3, which would give 3830 to 34470.

Let d and d_1 be the diameters, and l and l_1 the lengths of similar pillars, then as $d : d_1 :: l : l_1$, we have alternando $d : l :: d_1 : l_1$. From this proportion we may, it is evident, express the lengths in terms of the diameters, thus $l = rd$ and $l_1 = rd_1$; hence the strength varies as $\frac{d^{3.5}}{r^{1.63} d^{1.63}}$ to $\frac{d_1^{3.5}}{r^{1.63} d_1^{1.63}}$; or, making the divisions, as $\frac{d^{1.87}}{r^{1.63}}$ to $\frac{d_1^{1.87}}{r^{1.63}}$; that is, subtracting the index 1.63 from 3.55, we obtain 1.87.

In the numerical example quoted above, the length 20.166

To face page 102.



Pillars whose Length and Weight are the Same but their Strength varying as 1,2,3.

Kell Bros Lith, London.

is equal to 40×0.51; and 60.5 is also equal to 40×1.56, so that $r = 40$. In the Table below, we have r in the successive comparisons equal to 15, 20, 30, 40, 60, 30, and 20.

We must, therefore, in the particular case above, raise the numbers representing the linear ratio of the two pillars compared to the 1.87 power, in order that they may have the ratio of the breaking weights.

It is necessary to distinguish clearly between the linear ratio of the two pillars compared, and the ratio r of length to diameter in each particular pillar.

All the comparisons of the similar pillars should, however, be calculated separately as below, and an average taken: thus let t represent the index to which the numbers expressing the linear ratio of two pillars compared must be raised. We have, then, comparing lines 7 and 8 in the Table below,

$$1^t : 3^t :: 3830 : 28962;$$

or, dividing the two last terms by 3830,

$$1^t : 3^t :: 1 : 7.562;$$

and, therefore, $3^t = 7.562$, that is, $t \times \log 3 = \log 7.562$, or

$$t = \frac{\log 7.562}{\log 3} = \frac{0.8786367}{0.4771213} = 1.841;$$

and in like manner were obtained the other numbers which give the average of $t = 1.865$.

TABLE *showing the law of the strength of similar Pillars, or power of the lineal dimensions to which the Breaking Weights are proportional.*

No.	Value of r.	Diameter of pillars compared in inches.		Length of pillars compared in inches.	Breaking weight of pillars in lbs.	Powers of the dimensions.	
1 2	15	Pillars with ends rounded.	0.497 0.99	7.5625 15.125	5,262 19,752	1.908	Mean of the powers, 1.865.
3 4	20		0.76 1.52	15.125 30.25	9,223 32,531	1.819	
5 6	30		0.99 1.97	30.25 60.5	6,105 25,403	2.057	
7 8	40	Pillars with ends flat.	0.51 1.56	20.166 60.5	3,830 28,962	1.841	
9 10	60		0.50 0.997	30.25 60.5	1,662 6,238	1.9081	
11 12	30		0.51 1·02	15.125 30.25	6,764 21,844	1.6913	
13 14	20		0.50 1.022	10.083 20.166	8,931 31,804	1.8323	

[illegible]

[illegible] specimen, [illegible] pillar- [illegible]

[illegible] fracture [illegible] perpendicular to the axis is [illegible] we have

[illegible]

Hence [illegible]

The quantity within the parenthesis raised to the index [illegible] is computed upon the hypothesis of each of the pillars equally resisting crushing by that weight which causes fracture by flexure. The index of d, the division being carried out, is 0.9203, so that l varies very nearly as the first power of d; showing that, in the case of pillars equally loaded to resist crushing by the weight, the length to the diameter will be nearly in a constant ratio, or the pillars must be similar.

Had the strength been proportional to $\frac{d^4}{l^2}$, then l would have been exactly proportional to d; or, if the fractional index of d had been greater than that of l by 2, the same would occur.

In computing the gain of strength to be derived from any modification of the form of pillars, it becomes necessary to compare the breaking weights of the same quantity of metal in the various proposed forms,—the difference in this respect pointing out the gain of strength obtained; and, as an obvious simplification of the comparison, very easy to effect, the lengths of all must be made the same. As a standard of comparison, the cylindrical pillar is undoubtedly the best form to select,—a greater number of experiments having been tabulated on it than on any other.

It would, however, be almost impracticable to cast the pillar of proposed improved form, so that it should have the exact weight (after being turned to the same length) of any cylindrical pillar in the Tables. We must, then, take the weight and breaking weight of some one of the uniform pillars tabulated, and the weight of that of proposed figure, and, supposing this quantity of metal put into the form of a uniform pillar, compute its breaking weight from that of the pillar selected from the Tables,—the lengths of all being the same. A comparison between this calculated strength, and that derived from actual experiment of the pillar of new figure, shows at once the gain of strength derived from its peculiarity of form.

In order to make this calculation, we require to know the law of the strength of long uniform pillars, as dependent on their weight,—the length being constant.

Now, if the length be the same, the strength of uniform cylinders varies as $d^{3.5}$, and since the weight q of a pillar is as the square of the diameter, or directly as the area, d varies as $q^{\frac{1}{2}}$, whence the breaking weight varies as $q^{\frac{3.5}{2}}$, or as $q^{1.75}$ (this result applies only to pillars above a certain length: with flat ends, of a length not less than thirty times the diameter).

Thus in line V., Table § 124, p. 86, we have the weight of a pillar, 1.527 inches in diameter, and 60½ inches long, with ends rounded, equal to 28 lbs. 7 oz., and its breaking weight equal to 10,755 lbs. To calculate the breaking weight of a uniform, round-ended pillar of the same length, and of a weight of metal equal to that of proposed figure, say 26 lbs. 8 oz., we have, from the above law, the following proportion, reducing both to oz.:—

$$\overset{\text{oz.}}{455^{1.75}} : \overset{\text{oz.}}{424^{1.75}} :: \overset{\text{lbs.}}{10{,}755} : \overset{\text{lbs.}}{9530},$$

the breaking weight of a uniform cylindrical column, 60.5 in. long, and weighing 26 lbs. 8 oz.

It has been stated above, page 99, that long pillars of uniform thickness, with both ends alike, always break first at or near to the middle: this was the case even when they had discs upon the ends, to give the utmost firmness to their fixings. It seemed evident, then, that the pillar was always too weak there; Mr. Hodgkinson, therefore, proceeded to experiment on the effect of strengthening the middle of pillars; but as the ends could not be reduced at pleasure (which, if the diameter at the middle were unaltered, would be equivalent to a relative strengthening there), since they would be crushed without bending, in the manner of the shortest pillars, line XXVI., XXVII., § 125, the alternative was adopted of increasing the diameter in the middle, leaving that of the ends the same; and as it was best to make the experiments on pillars whose form was as simple as possible, models were made of the form of two frustrums of cones, the bases of which met in the middle of the pillar, the end diameter being 1 inch, and the middle in the different experiments 1¼, 1½, 1¾, and 2 inches,—the pillars being all of the same length, 60½ inches. The sides were formed rectilinear, and regularly tapering from the middle to the two ends. Fig. 30.

In a series of seven experiments on these pillars—enlarged at the middle, and having rounded ends—there was a gain in strength of about one-seventh, when compared with a uniform cylindrical pillar of the same weight. In those with discs cast on the ends—2 inches in diameter, and half-inch thick, turned flat — no gain in strength was obtained when the middle diameter was 1.30, compared with 1 inch at the ends; but when the middle diameter was 1.56 inch, compared with 1.06 inch at the ends, the gain was between an eighth and a ninth.

For example—a pillar with rounded ends, 1.04 inch at the ends, and 1.85 inches at the enlarged middle section, had a weight of 26 lbs. 8 oz., and a breaking weight of 11,069 lbs., the length being 60½ inches. If it should occur that among the tabulated results the strength of pillars of this length, and of ends the same in form, one was found whose weight was 26 lbs. 8 oz., then the direct comparison of the breaking weights gives at once the gain derived; if not, we must calculate as above. Now, we have already found by computation that the breaking weight of a uniform pillar, weighing 26 lbs. 8 oz., was 9530 lbs., and thus there has been a gain of 1539 lbs., the difference between these two, in favour of the pillar with enlarged middle section, and

$$\frac{1539}{11,069} = \frac{1}{7.192} \text{ or rather less than a seventh.}$$

131. A series of experiments was undertaken to determine whether the law of the strength of square prisms was identical with that of circular pillars, hitherto only tried, and which are in many points of view more convenient to employ in experimental investigations, especially as the square or rectangular bars always bend in the vertical plane passing through a diagonal.

TABLE *showing the Breaking Weights of long, uniform, square Pillars of Cast Iron, to a Force of Compression, the Ends being* ROUNDED.

Length 60.5 inches.

Low Moor Iron, No. 3.

	Dimensions of Section or Side of Square.	Mean Weight of Pillar.	Mean Deflection at Middle Height.	Mean of Breaking Weights.
	Inch.	lbs. oz.	Inch.	lbs.
1	0.7716	9 3¾	....	1269
2	1.0275	16 3½	0.44	3968
3	1.540	35 15½	0.42	16127

Comparing together, as in page 93, the ultimate strengths of the pillars whose sides are respectively

Inch.
0.7716 and 1.0275 gives $m = 3.9830$.
0.7716 „ 1.540 „ $m = 3.6789$.
1.0275 „ 1.540 „ $m = 3.4698$.

Mean $m = 3.7105$.

In the Table § 124, page 86, containing the experiments on long uniform cylinders of cast iron, ends rounded, we will, on computing, obtain as the mean of all the comparisons, excluding only those pillars whose length was not greater than fifteen times the diameter—$m = 3.736$; and therefore we may conclude that whether the pillar be circular or square, the power of the thickness to which the strength is proportional is nearly the same. And we may also conclude that the same value of m would be obtained for any other intermediate form of section, such as a hexagon, &c., &c.

It would appear to be otherwise, however, as regards the comparison of the breaking weights of circular, square, and triangular pillars, having equal weight of metal, and the same length. In the Appendix to his paper, Phil. Trans., 1857, p. 893, Mr. Hodgkinson gives in detail experiments on pillars of cast iron, Calder iron, No. 2, of a circular, square, and triangular section, the weight of each being nearly 150 lbs., and length 10 feet, with flat ends. The following is an abstract of the results :—

	lbs.
Circular pillars, mean,	55,299.
Square do., ...	51,537.
Triangular do., . .	61,056.

Each being an average of two experiments. The actual breaking weights were reduced in the proportion of the weight of the pillar to 150 lbs., which was the weight of one of the pillars, 10 feet long, and 2½ inches diameter.

One of the two triangular pillars, the average strength of which is given above, had the compressed part and place of fracture along the side, breaking with 57,332 lbs.; the other broke by the compression of the edge, showing a greatly increased strength, namely, an ultimate strength of 64,780 lbs. The iron in the edge, being harder than in the thicker parts, p. 8, § 13, would offer greater resistance to crushing than in them, and hence the strength of the pillar would be increased. Moreover, in this form of pillar, if the edge be the part crushed, it is at a greater distance from the centre of the section than in the others; and we have already seen, p. 77, § 110, that cast iron has a power of resisting crushing about six times greater than that to resist tearing asunder. From these and other causes the greater strength of a solid triangular pillar of cast iron, above those of the other two forms, may be inferred; a conclusion which may not be without importance in some applications of the metal.

Pillars of various other forms, and different modes of placing, have also been experimented upon,—the results from which will be given in the sections on Hollow Pillars.

132. The preceding experiments and deductions have been made upon solid pillars only, as these, on account of their greater simplicity, are not only more easy to be cast than hollow ones, but present fewer difficulties in experimenting upon, and in deducing conclusions. Solid pillars may be made as small as may be desired, but this is not the case with hollow ones. These require great care on the part of the moulder, and are consequently expensive. The first hollow pillars used in the following experiments were cast in a horizontal position; but it was found impracticable in this way to keep the core in the centre of the casting, and therefore several of the first pillars were defective, in having one side much thicker than the other, as well as having many air-bubbles in them. These causes tended to diminish the strength of the pillars; and will account for some results in the following Table, §§ 134, 135, being lower than they otherwise would have been. The circumstance of some of these pillars being much thicker on one side than the other, gave an opportunity of observing what falling off in strength arose from that cause, which is one to which cast iron pillars, as used in practice, must be very liable, though not so much so as in these experimental castings. They have therefore been tabulated with the others, as having some practical interest; and it is gratifying to find that a matter which would seem to destroy all confidence in a pillar does not produce a great reduction in the strength. The cause seemed to be this: in almost every case, where pillars were much thicker, in the middle, on one side than the other, the thin side was that which was compressed; and as cast iron resists fracture by compression, with (on the average) about six times the force that it resists tension, the pillar seldom gave way by compression, and therefore bore nearly as much as it would have done if of equal thickness on both sides.

If the thick side of the pillar had become that which was compressed, which seems as likely as otherwise, the strength would probably have been much decreased. But the circumstance of the thin side having been so often the compressed one, must apparently have arisen from some general cause.

The difficulty of obtaining good uniform hollow castings was met in the remaining pillars by having them cast vertically, and in dry sand; and this, together with great care, produced good castings, though the core could not always be kept in the centre towards the middle of the pillar. Fig. 29.

The greatest length of pillar to which the first apparatus was adapted (Phil. Trans., 1840) was about seven feet six inches; the length, namely, of the cast-iron box or frame in which the pillars were broken being a length half as much again as the greatest which was used in the experiments on solid pillars. As in the former experiments the five-feet pillars had, by a mistake in the commencement, been made half an inch too long, an anxiety to avoid error, however slight, led to the lengths of all the shorter pillars being made exact subdivisions of the longer; this entailed upon all the future experiments fractional lengths, which were introduced into the first series of experiments on hollow cylinders, in order to make them exact multiples of the former. The subsequent experiments (Phil. Trans., 1857) were extended to a length of ten feet, by using the larger and more powerful apparatus which had been constructed for experimenting on the resistance to compression of wrought iron tubes in the preliminary inquiries on the Conway bridge. Fig. 25.

The length of the hollow pillars, of the first series of experiments was, therefore, made 7 ft. 6¾ inches; and it was intended that their diameters should be as varied as possible, both for theoretical purposes and the application of their results to practice. There were difficulties, however, which militated against the execution of this plan. The external diameter of the pillars could not be reduced at pleasure; the internal must be large enough to admit a small hollow metallic tube, covered over with a thin stratum of loam, tied together with vegetable matter, the tube being perforated in every part with holes, and having its internal diameter large enough to allow the gas and steam to escape.

Through these causes, the smallest pillars which could be cast of the length of 7 ft. 6¾ in., were 1¾ inches external diameter, and 1⅓ internal. On the other hand, the experiments were bounded, in the largeness of the diameters, only by the power necessary to break the pillars. The apparatus by which the first series of experiments on hollow pillars was made consisted of a single lever, whose greatest multiplier was 7 or 7¼ to 1, and limit of strength the production of a pressure of about 18 tons; and in one of the experiments on hollow pillars (line 15 of the Table, p. 114), when an additional lever was added to the former, the weight applied was 22½ tons.

For these reasons, the variation in the diameters of the longest hollow pillars was necessarily small, particularly in those with flat ends, of which the largest external diameter was about 2¼ inches (line 15, Table, § 134), this requiring upwards of 18 tons to break it. In this the length of the pillar was 40 times the external diameter, and therefore might be considered as representing only the longest and slenderest pillars used in practice. In this matter, however, recourse was had very ingeniously to a truth which, as has been already mentioned, was discovered early in the investigation of the ultimate strength of solid pillars, namely, that the ratio of the strength of those with rounded ends to that of those with flat ends was nearly constant, and about as 1 to 3, and this in a range from the slenderest pillars down to a certain ratio of diameter to length.

It was, therefore, determined to pursue the experiments upon these hollow pillars (putting solid hemispherical caps upon the ends), increasing the diameters by small degrees, the length being the same, till they could be no further broken by this apparatus.

In this way the external diameter was varied from 1.74 inch to 3.36 inches, the former having sunk with 5711 lbs., about 2½ tons, and the latter with 50,477 lbs., or 22½ tons. In the largest pillars of the experiments, published 1840, line 14, 15, Table § 134, the ratio of the length to the diameter was reduced to 27 to 1. To pursue the matter further, it now became necessary to reduce the length of the pillars, because when that was done, the internal diameter could be reduced; and thus the external might be cast so small, that the strength of the metal in the pillars would be overcome by the power at command. Several of these pillars were turned outside and bored within, to make their dimensions as uniform as possible. By adopting pillars of smaller size than before, it was possible to render them as short, compared with the diameters, as might be wished, and thus obtain specimens of most of the varieties used in practice. A Table containing an abstract of these experiments on shorter pillars will be given in the discussion of the strength of pillars of length intermediate between those which are long enough to be destroyed by flexure, and those which are so short as to yield by crushing, and to which neither the laws of the strength of long pillars nor those of short specimens apply (page 82, § 118).

The following Tables, §§ 134, 135, contain an abstract of the results of experiments upon long hollow uniform cast iron cylinders, with an experiment or two in each upon solid cylinders of the same length to compare with them. The cylinders were always cast with great care, and were either perfectly straight or they were rendered so in the lathe, where their ends were turned flat and parallel. The pillars intended to be used with rounded ends were made about two inches shorter than the rest, or 7 feet 4¾ inches long, in order to allow room for the rounded caps upon the ends, as before mentioned, the whole length of the pillar with caps being 7 feet 6¾ inches.

133. "In solid cylinders of the same material, the strength to resist breaking varies according to some constant power m of the diameter, the length of the pillar and form of the ends being the same. Thus, the strength varies as d^m, the diameter being represented by d. In cylinders with flat ends, $m = 3.5$ nearly; and in those with round ends the same method of investigation (which it was not thought necessary to quote in detail) gave $m = 3.76$ nearly.

According to the mathematical theory already mentioned, page 85, the strength of a hollow cylinder to resist incipient flexure varies as the difference of the fourth powers of the external and internal diameters, or $d^4 - d_1^4$, where d and d_1 are the external and internal diameters.

It appeared probable that this ratio, modified by changing the indices to the results of the experiments on solid pillars, might agree with those on hollow ones. This was tried in the manner shown below, and found to answer very satisfactorily: so that we have

Strength as $d^{3.5} - d_1^{3.5}$, in pillars with flat ends.
Strength as $d^{3.76} - d_1^{3.76}$, in pillars with rounded ends.

If, then, from a measured external and internal diameter, and height of pillar, together with the breaking weight experimentally determined, as data, we calculate the breaking weight of a solid pillar of the same length, and one inch diameter, it will be found that this, so calculated, breaking weight is constant, provided the law of the strength of hollow pillars above enunciated is correct.

The strength of the solid pillar one inch diameter, with flat ends, may be expressed under the form $1^{3.5} - 0^{3.5}$; and hence, if W be the ultimate strength (known by experiment) of a hollow pillar, whose external and internal diameters are d and d_1, we have the strength W′ of a pillar, one inch in diameter, and the same length as the hollow pillar, by the proportion

$$d^{3.5} - d_1^{3.5} : 1^{3.5} - 0^{3.5} : : W : W',$$

so that

$$W' = \frac{W}{d^{3.5} - d_1^{3.5}}.$$

Thus, on the supposition of the breaking weight of any hollow pillar being proportional to the difference of the 3.5th powers of the external and internal diameters, the value of W′ should be constant. The results of many experiments gave an agreement as close as those of solid pillars, the weakest being but one-tenth below the mean.

And from this calculated value of W′ we can compute the breaking weight of a solid pillar, 1 ft. long and 1 inch in diameter, on the supposition of the strengths being inversely as the 1.63 power of the lengths. The results of this calculation are given in the columns headed "Value of the constant x."

As an example of the calculation, let line 19, page 115, be taken, we have then

$$W' = \frac{49.39}{3.532^{3.5} - 2.659^{3.5}} = 0.947 \text{ tons},$$

as the log of 3.532 is 0.54802, which multiplied by 3.5 is equal to 1.91807, answering to 82.81, in the same manner the internal diameter raised to the 3.5th power is 30.656, the difference being 52.154, which divided into the breaking weight gives W′ as above, the breaking weight of the solid pillar 10 ft. long and 1 inch in diameter. And again,

$$1^{1.63} : 10^{1.63} : : W' : x;$$

or since $10^{1.63} = 42.658$,

$$x = \frac{0.947}{42.658} = 40.4 \text{ tons},$$

as in column 8.

134. TABLE *showing the ultimate Resistance of Hollow Cylinders of Cast Iron.—Phil. Trans.*, 1840, *Part I.—Hollow Pillars*, ROUNDED *at Ends by Hemispherical Caps. Length* 90¾ *in.* = 7 *ft.* 6¾ *in.*

Low Moor Iron, No. 3.—Dry Sand.

	Diameters in Inches. External.	Diameters in Inches. Internal.	Area of Transverse Section in Sq. Inches.	Weight of Pillar, in lbs.	Breaking Weight, in lbs. in Tons.	Pressure per Sq. Inch, in Tons.	Deflection at Centre, in Inches.	Value of the Constant *x*, in Tons.
1	1.78	1.21	1.34	31	5585 = 2.49	1.85	0.49	Average value of *x* is equal to 13 tons.†
2	1.74	1.187	1.27	30¼	5711 = 2.55	2.00	0.48	
3	2.01	1.415	1.60	36½	8357 = 3.73	2.33	0.75	
4	2.33	1.70	1.99	46½	15089 = 6.74	3.38	0.72	
5	2.23	1.54	2.04	47	12389 = 5.53	2.70	0.69	
6	2.24	1.735	1.58	34¾	13341 = 5.96	3.79	0.38	
*7	2.24	1.58	1.98		13913 = 6.22	3.14	0.21	
*8	2.49	1.89	2.07	48¾	19855 = 8.86	4.28	0.52	
9	2.47	1.98	1.71	41	19003 = 8.48	4.96	0.62	
10	2.46	1.855	2.05	49	19147 = 8.55	4.17	0.65	
11	2.73	2.17	4.31	48	23963 = 10.70	2.48	0.70	
12	2.74	2.155	2.25	51¼	27883 = 12.45	5.53	1.10	
13	3.01	2.48	2.30	50¼	26707 = 11.92	5.19	0.75	
14	3.36	2.823	2.61	59½	40973 = 18.30	7.01	1.10	
15	3.36	2.63	3.44	77¾	50477 = 22.54	6.55	1.07	
16	Solid. 2.24		3.94	93	21281 = 9.50	2.41	0.25	

Experiments on larger Pillars.—Philosophical Transactions, for 1857, *Vol.* 147.

Low Moor Iron, No. 2.

Length, with rounded Caps, 10 feet 2 inches.

	External	Internal	Area	Weight	Breaking Weight	Pressure	Deflection	
19	3.532	2.659	4.25	130	45956 = 20.51	4.82	1.37	
20	3.555	2.693	4.23	128½	43537 = 19.44	4.60	1.40	

Lines 1, 2, 3, 5, 19, 20, in the above TABLES, §§ 134, 135, are on pillars identical, except in the form of the ends; the other lines of the same numbers have no correspondence.

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† This includes the results of the defective castings, and one very short pillar. It would otherwise have been above 14 tons.

135. Tᴀʙʟᴇ *showing the ultimate Resistance of Hollow Cylinders of Cast Iron.—Phil. Trans.*, 1840, *Part I.—Hollow Pillars*, ꜰʟᴀᴛ *at the Ends. Length* 7 *ft.* 6¾ *in.*

Low Moor Iron, No. 3.—Dry Sand.

	Diameters in Inches. External.	Diameters in Inches. Internal.	Area of Transverse Section Sq. Inches.	Weight of Pillar, in lbs.	Breaking Weight, in lbs. in Tons.	Pressure per Sq. Inch, in Tons.	Deflection at Centre, in Inches.	Value of the Constant *x*, in Tons.
1	1.78	1.21	1.34	31	17840 = 7.96	5.95	0.66	38.62
*2	1.74	1.187	1.27	30¼	16705 = 7.46	6.26	0.54	39.36
*3	2.01	1.415	1.62	36½	28353 = 12.57	7.86	0.25	41.76
4								
*5	2.23	1.54	2.04	47	40569 = 18.22	8.90		40.97
*6	1.76	1.18	1.34		16745 = 7.47	5.60	0.54	37.10
7	1.75	1.11	1.44	32	20957 = 9.36	6.50	0.55	44.81
8	2.04	1.46	1.60	35½	32413 = 14.47	9.04	0.52	45.13
9	2.01	1.368	1.70	37¾	30789 = 13.74	8.08	0.53	43.65
10	1.99	1.31	1.76	39	27067 = 12.08	6.86	0.90	38.25
	Solid.							
11	1.76		2.43	56	23179 = 10.35	4.26	0.65	38.71
12	1.72		2.32	53½	21995 = 9.82	4.23	0.65	39.81

Experiments on larger Pillars.—Philosoph. Trans. for 1857, *Vol.* 147.

Low Moor Iron, No. 2.

Length, 10 feet.

15	2.513	1.941	2.00	60¾	34804 = 15.54	7.77	1.00	44.27
16	2.520	1.903	2.14	64½	39166 = 17.48	8.17	1.16	46.92
17	3.021	2.354	2.82	89½	66259 = 29.58	10.49	0.93	45.21
18	3.035	2.354	2.88	88	69932 = 31.22	10.84	0.91	47.09
19	3.532	2.659	4.25	130	110649 = 49.39	11.62	1 00	40.40
20	3.555	2.693	4.23	128½	114479 = 51.11	12.08	0.80	41.40
21	3.804	3.17	3.47	106⅝	107403 = 47.95	13.82	0.48	40.52
22	3.819	3.211	3.84	99⅞	95519 = 42.64	12.77	0.24	36.73

Solid uniform Pillars, 10 *feet long,*

Low Moor Iron, No. 2.

23	2.553		5.12	159⅜	55973 = 24.99	4.88	1.23	40.09
24	2.539		5.07	159	55973 = 24.99	4.93	0.79	40.87

Average, 42.35 tons.

136. In the Tables given above, § 134, 135, are arranged the results of experiments on long hollow pillars; page 114 containing those rounded at the ends, and page 115 those turned flat and perpendicular to the axis, and in each are included the last experiments on columns, Phil. Trans., 1857, approaching nearly 4 inches external diameter, and 10 feet in length, with breaking weights of nearly 50 tons. The pillars are tabulated very nearly in the order of their strength, both through those of 1840 and of 1857.

The first column in each Table contains the number of the experiment; lines 1, 2, 3, 5, 19, 20, in § 134, being the same pillars as in the like numbers in § 135.

The second column, in giving the internal diameter of the pillar, gives it as calculated, as well as measured. This was done because there was a great difficulty in measuring correctly the internal diameters, which were in some degree irregular. The weight of a solid pillar of a certain diameter and given weight being known from a mean of three experiments, and the weights of the hollow cylinders of the same length as the solid pillars, together with their external diameters, being also known, from these data the internal diameters of all the cylinders were computed, and a mean between the calculated and the observed results was taken for the internal diameter. This extra trouble was taken, as it was thought desirable to avoid every possible inaccuracy in experiments which, from the labour and expense attached to them, were not likely soon to be repeated. Thus, if on trial it is found that a solid pillar, 10 feet long, and 2½ inches in diameter, weighs 150 lbs., and the external diameter of a hollow cylinder of the same length, measuring 4 inches, has a weight of 120 lbs., from these data we may determine the internal diameter x, and the thickness of the ring as follows. The weight of a solid cylinder, 4 inches diameter, is found from that of 2.5 inches, by the proportion

$$2.5^2 : 4^2 :: 150 : 384 \text{ lbs.};$$

and if we deduct the weight of the ring, 120 lbs., from this, we evidently have the weight of a solid pillar whose diameter is the internal diameter of the hollow cylinder: thus,

$$384 - 120 = 264 \text{ lbs};$$

and again,

$$150 : 264 :: 2.5^2 : x^2;$$

so that

$$x = \sqrt{\frac{2.5^2 \times 264}{150}} = 3.366,$$

and the thickness, which is half the difference of the diameters, is equal to

$$(4 - 3.3166) \div 2 \doteq 0.3417 \text{ inch}.$$

The internal diameter in line No. 9, Table page 115, is given as 1,368 inches; by measure it was 1.330, and by calculation 1.407, the mean being as above: this appears to be the highest difference recorded between the measured and calculated diameters.

The third column contains the area of the transverse section of the pillars, showing the gradual increase of the pillars in the successive experiments, and will be interesting as showing the great difference between the ultimate crushing strength of such area, and the actual breaking weights of the long hollow pillars, found by multiplying the area by 42 tons, as the mean crushing strength.

The fourth column gives the weights of the pillars, and like the third shows the increasing magnitude of the experimental pillars; and by comparing the weights and breaking weights, with the weights and breaking weights of solid pillars, shows the advantage of throwing the material into the form of a hollow pillar: thus in line 24 of the Table page 115, we have a solid pillar weighing 159 lbs. broken with 25 tons, and in line 22 a hollow pillar of 100 lbs. weight, or less than two-thirds, broken with 42.64 tons.

The fifth column contains what is denominated the breaking weight, but which, in some cases, means only that weight which was sufficient to overcome the greatest resistance of the pillar. This was frequently attained when a long plllar had been bent through a space not exceeding its radius. From the first experiment on long hollow pillars with rounded ends, it was evident that so little flexure of the pillar was necessary to overcome its greatest resistance (and beyond this a smaller weight would have broken it), that the elasticity of the pillar was very little injured by the pressure, if the weight was prevented from acting upon the pillar after it began to sink rapidly, through its greatest resistance being overcome. By attending carefully to this point, the pillar was frequently saved uninjured for another experiment. The small flexure it had got was straightened, and the rounded ends being removed, the pillar, with its flat ends well bedded against the crushing surfaces, was broken, and thus the comparative results in lines 1, 2, 3, 5, 19, 20 in Tables §§ 134, 135, p. 114 and 115, were recorded.

The sixth column is the quotient obtained by dividing the area by the breaking weight; and although it is known, § 138, that only part of the transverse area is under compression in long pillars, it was yet deemed useful to tabulate the results; for they show, in some degree, how small a part of the ultimate crushing resistance is borne by flexible columns, and also how considerable is the increase with the better forms of hollow pillars.

The seventh column gives the ultimate central deflection, and shows (with considerable irregularities, however), that it increases in some degree with the radius.

The eighth column is the strength of a solid pillar 1ft. long and 1 inch diameter, as computed from the breaking weights and dimensions in each particular experiment, on the supposition that the strength is as the diameter raised to the index 3.5, or, if hollow, the internal subtracted from the external diameter, each raised to the same index above, and as the inverse length raised to the index 1.63.

137. It has been already shown in pages 106, 107, that a solid cylindrical pillar, with rounded ends, and enlarged in the middle of its length, was stronger than a uniform pillar of the same weight. There seemed little doubt that the same would be the case in hollow pillars, or that a hollow pillar with an enlarged middle, having the metal everywhere of uniform thickness, would be stronger than a hollow uniform pillar of the same weight and thickness of metal. To form these pillars with much regularity in small experimental castings, would have been difficult; but some idea of the influence of increasing the middle might be obtained by reducing the thickness of the uniform pillars, near to the ends, leaving the middle the same. The pillars in lines 7 and 8, with an asterisk, page 114, in which the ends were rounded, were reduced in the following manner. The pillar was reduced to half its thickness of metal, in a band two or three inches broad, near to each end, and to three-fourths of its thickness, in a similar band, about half the distance between the middle and each end Fig. 32. These pillars did not break in the reduced parts; and from the results it would seem doubtful whether the strength had been in any

degree lessened by the partial reduction of thickness. If this reduction had been carried uniformly on, decreasing from the ends to the middle of the pillar, leaving it the same there, the pillar would have had but three-fourths of the metal in it, and its strength would perhaps have been but little decreased.

In the Table page 115 the lines, with an asterisk, 2, 3, 5, 6, were reduced in the manner above described, by having portions cut away from each. These never broke in the reduced parts first, except in one instance, line 6, and that broke half-way between the middle and the ends where it had been reduced somewhat more than one-fourth, which was the intended reduction. It would appear that the mean of those not reduced was about = $\frac{1}{13}$th stronger than those which had been so, leaving it doubtful whether reducing the thickness towards the ends in hollow pillars with flat ends, effects any saving in the metal.

The average "Value of x" in the experiments of 1840, Table page 115, the pillars having flat ends, is 40.75, very little short of three times that of those with round ends, page 114, namely 14 tons.

It will be perceived that the breaking weights in lines 19 and 20, Table § 134, are greater than one-third of the same lines and same pillars, (but flat at the ends) in Table § 135, arising in a great measure from crushing of the plates, between which they were compressed, and from that also of the rounded ends of the pillars themselves, through the great weight required to break them.

The pillars, lines 21, 22, page 115 were cast 4 inches external diameter; and this dimension being deemed beyond the power of the apparatus to break, they were turned on the outside to reduce them. The great reduction in strength produced by taking away the external crust shows that, to ornament a pillar, it would not be prudent to plane it. The pillar in line 22 had, however many flaws in it.

138. The fractional indices for the powers of the diameter and length, namely, 3.5 and 1.63, to which the ultimate strength of long pillars is proportional, as obtained by experiment, do not, in reality, contradict the results of theory. It has been mentioned, §§ 123; 126, that the objects in view of the experimenter and the mathematician were different at starting: the latter supposed the material of the pillar to be incompressible, or, in other words, that in bending, no part of the transverse section of the pillar was under compression; all was under tension, and resisting lateral flexure, Now, in the experi-

ments tabulated, page 87, even the very slender columns were found, in the act of breaking, to have had a portion of the transverse area compressed; and as the ratio of the length of the pillar to the diameter was decreased, that is, as it became shorter in reference to the diameter, it was found that the part sustaining tension, compared with that in compression, successively decreased, and finally became much the smaller. Fig. 33. The following statement is selected from pillars in Table, § 125, page 87:—

	Ratio of Ver-Sines of Segments under Tension. Compression.	Ratio of Length to Diameter.
Line V.,	$3\frac{1}{4}$ to 1,	40 to 1.
,, X.,	$1\frac{2}{3}$ to 1,	30 to 1.
,, XIV,,	$1\frac{1}{3}$ to 1,	26 to 1.
,, XX.,	1 to 1,	$15\frac{1}{2}$ to 1.
,, XXIV.,	$\frac{4}{5}$ to 1,	10 to 1.
,, XXV.,	$\frac{2}{3}$ to 1,	$7\frac{1}{2}$ to 1.

Thus the breaking weight must depend on the resistance to crushing, as well as on that to flexure.

These numbers are in the ratio of the versed sines of the arcs of the parts under tension and compression respectively.

The neutral line was indicated in some cases by the depth of the wedge, which broke out on the compressed side, and in others by a crack separating the two surfaces. Though, strictly speaking, the neutral line on the transverse section of a cylindrical pillar is a curve, and not a straight line, the above ratios have been adapted, so that it may be considered to be such. It is plain, therefore, that the actual circumstances of loaded pillars are very different from that assumed in Euler's theory; and, so far from there being no compression, we find that, in cast iron, the transverse area under tension is, in the case of very short pillars, much less than that in compression.

It is equally important to observe that, in proportion as the circumstances of any experiment approached to that of the theoretical hypothesis, so the fractional indices of the inverse length and of the diameter approached the limits of 2 and 4 respectively. Thus, in pillars with rounded ends, § 124, comparing the strengths of those of 60.5 inches long with those of

30.25, 15.125, 7.5625, 3.78125 inches long,
we have $n =$ 1.914, 1.867, 1.734, 1.681,
and 539, 1,904, 5,262, 15,107, breaking wts. in lbs. respectively, the diameter being 0.50 inches in each case.

In the pillars with flat ends, § 125, comparing the strengths of those 60.5 inches long with those of

30.25, 20.166, 15.125, 12.1 inches long,
we have $n =$ 1.843, 1.682, 1.550, 1.424,
and 8,811, 15,581, 21,509, 24,287, the breaking weights in lbs., the diameters being 0.77 inches.

Though the value of n in the above series never rises so high as 2, yet we see that it approaches to it as the breaking weight becomes smaller, and the part of the transverse section in compression smaller also, which facts, combined with the regular increase of n as the breaking weight is diminished, show that 2 is the value to which n would approximate if the breaking weight were indefinitely small, or the body perfectly incompressible.

In like manner the power of the diameter m approximates, but never rises up, to 4 in pillars so long with respect to their diameter that the breaking weight is very small: that is, in pillars whose breaking weight was so small that it produced little or no compression on the transverse area, then the index will arrive at the theoretical value of 4. The Tables, pages 93, 94, 95, point out very clearly the gradual increase of m as the breaking weights compared are less. So that we may conclude that $\frac{d^4}{l^2}$ represents the limit to which the fractional indices both of d and l tend; and in this, as in all other cases, the discrepancy between practice or experiment on natural results, and theory, resolves itself into a discrepancy between the primary hypotheses of the mathematician and the actual causes and conditions at work, which are always more numerous and complex than the assumptions of the philosopher.

In this particular case of cast iron pillars, the compressibility of this material was not taken into account; and the existence of a neutral line, dividing the transverse section into parts, acting with a totally different manner of resistance, one part being in compression, and the other in tension, and which line, moreover, changed its place as the pressure increased, were unsuspected, and found no place in the data. It has been already mentioned, page 88, that the theoretical law of the strength, $\frac{d^4}{l^2}$, was, notwithstanding, of great service in arriving at the true value of the indices; and in one material, timber, we find the power of the diameter so near 4, that it may be assumed to be equal to that number. Indeed, experiment and theory mutually confirm each other; since, in proportion as the conditions of any experimental pillar approach the suppositions of theory, so the fractional indices approximate to the value $\frac{d^4}{l^2}$. If, then, the power of the diameter to which the strength is proportional range between 4 and 3.12 (which is the lowest observed), and that of the inverse length from 2 down to 1.4, what is the meaning of the law $\frac{d^{3.5}}{l^{1.63}}$ and $\frac{d^{3.5} - d_1^{3.5}}{l^{1.63}}$ for solid and hollow pillars, respectively, which have been determined upon in the previous pages? Manifestly each is only an average of the results of experiments taken between limits which, excluding all that were too short in proportion to the diameter, yet included the most usual proportions found in practical construction.

In order to facilitate the calculations necessary in designing cast iron pillars, a Table of the 3.5th powers of the more usual diameters is given after the Practical Application of Cast Iron in Compression; and also the areas of the circles with those several diameters are added in the same line.

139. We are thus led to the consideration of the third set of distinct laws which govern the strength of columns, already mentioned in § 118, pp. 82, 83, namely, that which obtains in the lengths between about 5 diameters, below which the strength is constant, and about 30 diameters, above which it diminishes inversely as the 1.63 power of the length. Does the law of the strength, then, change *per saltum* at these limits? Such a conclusion would be directly contrary to all analogy: the lines of demarcation mentioned are mere arbitrary divisions, but on either side of which the strength alters in a marked degree.

The following Table, taken from the shorter pillars in Table, § 125, shows the falling off in the calculated value of the constant x, the lengths being in every case less than 30 to 1:—

140. TABLE *showing the falling off in the calculated Value of the Constant x, in Pillars shorter than* 30 *times the Diameter.—Ends* FLAT.

Diameters.	Lengths.	Ratio of Diameter to Length.	Breaking Weights.	Per Sq. Inch of Section.	Value of x.
Inches.	Inches.		lbs.	Tons.	lbs.
0.77	20.166	1 : 25.95	15,581	14.7	92,224
0.50	12.1	1 : 24.20	7,195	16.4	85,467
0.50	10.083	1 : 20.166	8,931	20.3	77,817
1.022	20.166	1 : 19.732	31,804	17.3	71,154
0.775	15.125	1 : 19.516	21,059	19.9	77,141
0.785	12.1	1 : 15.414	24,287	22.4	46,207
0.50	7.5625	1 : 15.125	11,255	25.6	60,133
1.00	15.125	1 : 15.125	40,250	22.9	59,516
0.768	10.083	1 : 13.129	25,923	25.0	49,224
0.777	7.5625	1 : 9.733	32,007	30.1	35,757
0.50	3.7812	1 : 7.562	17,468	39.7	28,725
0.52	2.00	1 : 3.846	22,867	48.1	11,080
0.52	1.00	1 : 1.923	24,616	51.74	

141. The above values of x have been calculated from the earlier formulæ of Mr. Hodgkinson, which gave an average of 98,922 lbs. for flat-ended pillars, above 30 diameters long. The value of x in the above Table, instead of being constant, evidently falls off rapidly, as the ratio of the length to the diameter decreases.

Taking the average of the three which have the ratio of 15 to 1 (half that of 30 to 1), we obtain the mean, 55,285, which is somewhat greater than 49,461, the half of the general mean.

Again, in pillars whose length is 7.56 times the diameter (one-fourth of 30 to 1), the corresponding number is reduced to 28,725, a little more than one-fourth of the general mean, which is 24,731.

And those whose length was about 20 times the diameter (two-thirds of 30 to 1), had a mean of 75,371, also greater than 65,948, which is two-thirds of the general mean.

Now, in all these the length of the pillar is obviously the same proportion of the length of a pillar 30 times the diameter, which the "Value of x" is of the general mean. Thus, a pillar 15 to 1 is half the length of one of the same diameter 30 to 1, and its corresponding value of x is half the general mean. So, also, a pillar 7.56 diameters high is one-fourth of one 30 diameters high, and its calculated strength but one-fourth of the general mean. The pillars whose form was 20 to 1 are in length two-thirds of one 30 to 1, and in this also the formula gives somewhat more than two-thirds of the general mean. We might, therefore, obtain a rough approximation to the strength of short pillars by calculating by the formula before used (that is, with the constant 95,000, and $d^{3.5}$ and $l^{1.63}$), the strength of the pillar, and then taking such a portion of this calculated strength as the length of the pillar is of what it would have been if it were 30 times the diameter,—this simple rule giving the strength somewhat under the real strength, as appears from these and other instances in the Table.

142. Another mode of considering the matter will, perhaps, throw some light upon the reason of the conclusion arrived at in the last section, and furnish better means of estimating the strengths. If we compare the several corresponding numbers in the columns of weights per square inch and "Values of x" in the Table above, we find, amidst many anomalies, that the weights increase in some such proportion as the values of x decrease; and the latter become very small where the pressure approaches to the crushing force.

Considering, then, the pillar as having two functions, one to support the weight and the other to resist flexure, it follows that, when the pressure necessary to break the pillar is very small (i. e. in reference to the ultimate crushing resistance of its transverse area), on account of the greatness of its length compared with its lateral dimensions, then the strength of the whole transverse section of the pillar will be employed in resisting flexure; and when, from the shortness of the pillar, the breaking pressure rises to one-half of what would be required to crush a short specimen of same area, then one-half only of the strength may be considered as available for resistance to flexure, whilst the other half is employed to resist crushing; and when, through the shortness of the pillar, the breaking weight is so great as to be nearly equal to the crushing force, we may consider that no part of the strength of the pillar is applied to resist flexure.

We may, therefore, separate these effects by supposing that there is taken from the transverse area of the pillar as much as would support the pressure as a crushing force on a short specimen, and consider the remainder as resisting flexure to the degree indicated by the formula for long pillars with flat ends, namely, $\frac{d^{3.5}}{l^{1.63}}$. If, for simplification, we suppose the transverse area to be rectangular, the strength will be as the breadth (or longest side) into the cube of the other; and therefore we may take from the area of the pillar, as above mentioned, by reducing its breadth, since the strength is proportional to the breadth. Suppose then

c = the force which would crush without flexure a short specimen of the material having the same transverse area as the pillar.

d = the utmost pressure the pillar, as flexible, would bear to break it, without being sensibly weakened by crushing.

b = the breaking weight, as calculated by the formula $42.35\frac{d^{3.5}}{l^{1.63}}$.

y = the actual breaking weights (as determined by experiment) of the shorter pillars, of length intermediate between those which break with little or no crushing, and the short specimens which yield altogether by crushing.

Now, we have seen that the breaking weight of the shorter pillars depends both on the resistance to crushing and on that to flexure, and therefore a formula for their strength must include both c and b, and also d, unless expressed in terms of the other quantities.

143. If then we supposed a portion equal to what would just be crushed by a pressure equal to d to be taken away from the transverse area of the pillar, what is the value of the crushing pressure of the remainder of the area? It must evidently be the difference between the crushing force of the whole area and that of the part supposed to be removed as above, so that it may be expressed by $c - d$.

And again, y being the actual breaking weight distributed uniformly over the whole transverse area, $y - d$ must represent the weight actually laid upon this remaining area above mentioned. Whence, if we call the remaining area, of which $c - d$ is the crushing strength, unity, the part of this remaining portion of the pillar which has to resist crushing may be expressed by the fraction $\frac{y - d}{c - d}$, and of course that which has to sustain flexure will be $1 - \frac{y - d}{c - d}$; simplifying this expression, by multiplying by the denominator and reducing, we have

$$1 - \frac{y - d}{c - d} = \frac{c - y}{c - d},$$

and by this fraction we must multiply the calculated breaking weight b (which has been derived from $42.35 \frac{d^{3.5}}{l^{1.63}}$, applied to the short pillars we are considering), to obtain y; so that the two parts are expressed by the fractions

$$\frac{y - d}{c - d} \text{ and } \frac{c - y}{c - d},$$

the former having to resist compression, the latter to sustain flexure.

We may deduce from what has been shown in previous remarks, that when the breaking weight in any experiment was about one-fourth of the weight which would have crushed the whole pillar without flexure, then the result of calculating its strength by the general formula for long pillars needed modifying.

144. If there were no fixed pressure at which pillars in breaking suffered a marked diminution of their calculated strength, but that there was a regular falling off with all weights from the least necessary to break a long pillar up to such as would crush it, then d might be taken equal to 0, and the two fractions just given become $\frac{y}{c}$ and $\frac{c - y}{c}$, by which last we must multiply the calculated breaking weight b to obtain y, that is,

$$b \times \frac{c - y}{c} = y; \text{ or } \frac{bc}{b + c} = y.$$

But as there is no such pressure, we proceed thus,—the strength of the pillar, if rectangular, may be supposed to be reduced by reducing either its breadth or the computed strength of the whole to the degree indicated by the fraction $\frac{c-y}{c-d}$. In circular pillars the first mode is not strictly applicable; but we obtain a near approximation to the breaking weight y by reducing the calculated value of b in that proportion. We have, therefore,

$$b \times \frac{c-y}{c-d} = y,$$

so that

$$bc - by = cy - dy,$$

and

$$y = \frac{bc}{b+c-d}.$$

And as it may be shown that about one-fourth of the crushing weight was the utmost that a flexible pillar could have as its breaking weight without the material being crushed, and the breaking weight reduced in consequence, the above expression becomes,

$$y = \frac{bc}{b+\frac{3}{4}c}.$$

The value of y above is compounded of two quantities, b the strength obtained from the formula for long flexible pillars, and c the crushing force of the particular transverse section.

The following Table, giving the dimensions and breaking weights of eleven short solid pillars with flat ends, together with the calculated values of b, c, and y, will show the degree of approximation that the ultimate strength calculated by this formula bears to the real strength given by experiment.

145. TABLE *showing the actual Breaking Weights of Solid Cast Iron Pillars shorter than* 30 *times the Diameter, and the calculated Breaking Weights, by the proposed Formula — Ends* FLAT.

Diameters.	Length of Pillar.	Value of $b = 42.35 \frac{d^{3.5}}{l^{1.63}}$.	Value of Crushing Force, c.	Actual Breaking Weight, y.	Value of $\frac{bc}{b + \frac{3}{4}c}$.
Inches.	Feet. Inches.	lbs.	lbs.	lbs.	
0.50	1.008 = 12.1	8,327	21,559	7,195	7,328
0.50	0.840 = 10.083	11,353	21,559	8,931	8,872
0.50	0.630 = 7.5625	18,515	21,559	11,255	11,508
0.50	0.315 = 3.7812	60,155	21,559	17,468	16,992
0.777	1.681 = 20.166	16,713	52,064	15,581	15,604
0.775	1.260 = 15.125	27,005	51,797	21,059	21,241
0.785	1.008 = 12.1	41,300	53,142	24,287	27,043
0.768	0.840 = 10.083	52,096	50,865	25,923	29,363
0.777	0.630 = 7.5625	88,547	52,064	32,007	36,130
1.022	1.681 = 20.166	44,218	90,074	31,804	35,631
1.00	1.260 = 15.125	66,746	86,238	40,250	43,797

146. It will be observed that the calculated strengths agree moderately well with the real ones, showing that the resistance to crushing is an element of the strength of *short* flexible pillars at least. Another Table is given in the Phil. Trans., 1840, page 448, Table X., containing the results on uniform hollow cylinders shorter than 30 times the diameter; and the accordance between the actual and calculated breaking weights in these was in the same degree as in the former.

147. In the following very important and interesting Table are shown the strengths of different forms of hollow cylinders compared with the uniform hollow cylinder.

The first two pillars were from the same model. They were about 2¼ inches in diameter in the middle, and 1¾ inch near to the ends, which had strong discs upon them, turned flat, to give them a firm bearing; the pillars were hollow, fig. 34*a*.

The third and fourth pillars are two hollow pillars, 2¼ inches external diameter at the bottom, and 1¾ at top nearly. They tapered regularly from the bottom to the top, where there was a disc to increase the stability of fixing, fig. 34 *b*.

The fifth was on a uniform pillar, whose transverse section was of the form fig. 34, *c*, three inches across and half-inch thick, the ends being rounded, that the force might pass through the axis.

The sixth was a uniform pillar of the section **I**, and of the same area as the last, fig. 34, *d*.

Experiments 7 and 8 show the strength of solid pillars when the pressure is applied to some point intermediate between the centre and the circumference; and exhibits a great falling off from what it would have been if the pressure had been through the axis, as in round-ended pillars from the same model, § 125, p. 87, fig. 34, *e* and *f*.

Experiment 9 is on a solid pillar so fixed that the pressure is in the diagonal, fig. 34, *g*.

The long pillars whose strength has been given in the former Tables, and the following, page 131, comprise most of the forms at present used in practice.

The mean breaking weights of the columns in lines 1 and 2 is 28,079. From a mean between experiments, lines 3, 8, 9, § 135, page 115, it appears that a hollow uniform pillar, 2.02 inches external diameter (nearly the mean of 2.24 and 1.73), and weighing 36½ lbs., had a breaking weight of 30,518 lbs., which seems to show that the uniform cylinder is the stronger.

The two pillars in lines 3 and 4 had a mean external diameter at base of 2¼ inches, and nearly 1¾ at top, tapering regularly to the top, where there was a disc to increase the stability of fixing. The mean weight of these was 36.8 lbs., and the mean breaking weight 29,488 lbs. But we have just seen that a uniform hollow pillar, 2.02 inches diameter, and 36½ lbs. weight, would require 30,518 lbs. to break it. Therefore, these pillars, like the former, are weaker than uniform hollow pillars.

The pillar in line 5 was uniform, and had rounded ends. The section of the form fig. 34, *c*, 3 inches across the ribs, and 0.48 inch thick. Now, if we refer to line 13, § 134, p. 114, we find a hollow pillar, 3.01 inches diameter, and 50¼ lbs. weight, broken with 26,707 lbs.; but though this pillar, cruciform in section, weighed 62 lbs., it broke with 17,578 lbs. Whence we see that pillars of this form, which is nearly that of the connecting rod of a steam-engine, are very weak compared with uniform hollow cylinders; the strength of which being as the 1.714th power of the weight nearly (pages 105, 106); a hollow cylinder, if of the same weight, that is 62 lbs., as the pillar above, would have required 38,345 lbs., or more than twice 17,578, to break it; thus,

$$\left.\begin{array}{rl} 50.25^{1.714} : & 62^{1.714} \\ \text{or } 823.67 : & 1182.6 \end{array}\right\} :: 26,707 : 38,345.$$

The pillar in line 6 had a section of the form **I**, of the same area as last; the parallel bars were 3 inches, the cross piece $2\frac{1}{2}$ inches long. This was considerably stronger than the last, but much weaker than a hollow cylinder of the same weight. Using the same principles as above, we obtain for this form of pillar.

$$\left.\begin{array}{rl} 50.25^{1.714} : & 63.5^{1.714} \\ 823.67 : & 1230.14 \end{array}\right\} 26,707 : 39,887.$$

Which fourth term is greater than 29571 by somewhat more than one-third.

Lines 7 and 8 show the strength of solid pillars, when the pressure is applied at some point intermediate between the centre and the circumference; and exhibits a great falling off. Comparing the first of these, whose breaking weight is 13,195 lbs., with that of line VII., Table § 124, page 86, of the same dimensions, the breaking weight being 23,551 lbs., we perceive a great falling off in strength. And again, the pillar in line 8, which had a breaking weight of 14,697 lbs., is also much weaker.

The mean of lines 9 and 10 gives a breaking weight of 6512.5 lbs.; and comparing this with line X., in Table, §§ 124, 125, pp. 86, 87, which are of the same dimensions very nearly, we have 6105 lbs. for the round-ended, and 20,310 for the flat-ended pillars, and thus the force directed diagonally through a column reduces its strength to a third of what it would support if with a flat and true bearing, showing the importance of the fixing of the base of columns on the surface of the stone or concrete foundation, and also of a true packing between the top of the cap of the pillar and the under side of the beam resting on it.

148. TABLE *showing the ultimate Strength of Pillars of various Forms and different Modes of placing.*

		Deflection.	Weight of Pillar.		Breaking Weight.
		Inches.	lbs.	oz.	lbs.
1	Round hollow pillar, regularly tapering from the middle to the ends, like the frustrums of two cones whose bases joined in the middle of the pillar. External diameter:— „ at middle, . . 2.24 inches. „ at ends, . . . 1.73 „ Internal diameter, . . . 1.63 „ Length, 7 feet 6¾ inches, ...	0.54	36	0	28,667
2	Pillar from same model as No. 1, ...	0.65	36	4	27,491
3	Round hollow pillar, tapering regularly from the base to the top, as the frustrum of a hollow cone. Disc at top 4.44 inches diameter. External diameter:— „ at bottom, . . 2.22 inches. „ at top, ... 1.70 „ Internal diameter, . . . 1.61 „ Length, 7 feet 6¾ inches, ...	0.87	39	0	32,413
4	Same form as No. 3. Thickness 0.25 inches. External diameter:— „ at bottom, . . 2.32 inches. „ at top, ... 1.75 „ Length 7 feet 6¾ inches, ...	0.80	34	9	26,563
5	Transverse section, fig. 34, *c*. Length, 7 feet 6¼ inches, ...	0.45	62	0	17,578
6	Transverse section, I. Length, 7 feet 6¾ inches, ...	0.25	63	8	29,571
7	Solid uniform cylinder, pressure directed to pass through a line half-way between centre and circumference. Diameter, ... 1.95 inches. Length, 60.5 inches, ...	1.30	45	14	13,195
8	Solid uniform cylinder, same as last; force directed to pass ¼th of radius from circumference. Diameter, ... 1.96 inches.	1.48	46	5	14,697
9	Solid uniform cylinder; force directed diagonally through opposite corners. Diameter, ... 0.97 inches. Length, 30½ inches, ...	. .	5	13	5750
10	Same as last. Diameter, ... 1.02 inches.	. .	6	4¼	7275

PRACTICAL APPLICATIONS AND EXAMPLES—CAST IRON IN COMPRESSION.

149. The practical applications of cast iron under direct compressive strains are very numerous, and it is under this head that its chief utility as a constructive material is found. Its weakness under tensile forces is amply compensated here. We find it almost exclusively employed for pillars both solid and hollow. Notwithstanding the modern rivalry of wrought iron, it is even now often used in the ribs of cast iron arches of segmental form, especially in spans too great for bridges of stone or brick masonry, at least with the small degree of rise or versed sine which may be used in this material, as compared with what would be prudent to employ in brick or stone. It is used with advantage in the jibs of cranes. In struts to tension rods for iron roofs, and timber beams. In hollow pipe struts for the lateral stiffening of girders, ribs of cast iron arches, and beams of steam-engines when divided: a wrought iron tie-bar passing through the hollow, with screw bolts on the exterior faces.

In these examples, and in any material under compression generally, it is not of the same importance to compare the ultimate strength, with the ultimate strength in combination, as it was in applications of material to resist tensile forces, the action of the compressing forces being to unite more closely the surfaces of contact, which should be made as true and flat as possible. It is the danger of lateral flexure which distinguishes the effects of compression from those of tension, and which must be so carefully guarded against.

150. Cast iron in the form of hollow pillars is a very common and very appropriate application of the high resistance to compression possessed by this material. Indeed, when the diameter is small, and the height and load to be supported not very great, the solid pillar is often made use of in construction.

An inaccuracy of language, and confusion of ideas on the comparative strength of hollow and solid pillars, may often be observed. It is said, for instance, that a hollow is stronger than a solid pillar; such is not the case absolutely, the solid is stronger than the hollow of the same external diameter. If a solid and hollow pillar of the same length have the same external diameters, and we compare the ultimate strengths of each, and also compare the areas of their transverse sections, we shall find that the first ratio is far greater than the second, which shows the true advantage of hollow pillars in the economy of material.

For example, comparing the relative strengths of two pillars, each 10 ft. long, the solid having 6 inches diameter, the hollow also 6 inches external diameter, the internal being 4.8 inches. From these data we find that the transverse area of the solid pillar is 28.27 sq. inches, and that of the hollow 10.18 sq. inches.

On the other hand, the ultimate strengths are $42.35\frac{6^{3.5}}{10^{1.63}}$ $= 525$ tons, and $42.35\frac{6^{3.5} - 4.8^{3.5}}{10^{1.63}} = 284.72$ tons. But the ratio of the areas, or quantities of metal, would only give

$$28.3 : 10.2 :: 525 : 189 \text{ tons},$$

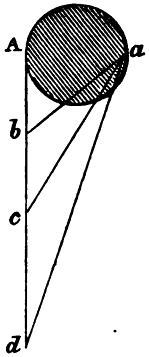
or about 96 tons less than the real breaking weight.

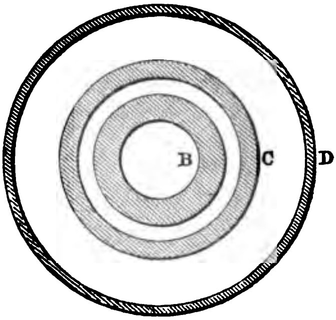
If the interior diameter had been 4.9 inches, we have the area of the transverse section of the ring of metal, 9.427 sq. inches, or one-third of that of the solid pillar, and the ultimate strengths will be, the solid pillar as above, 525 tons, and the hollow, $42.35\frac{529.1 - 260.43}{42.658} = 267$ tons, but little more than half the former; so that, the external diameter being the same in each, two such hollow pillars would support as much as the one solid pillar, though containing but two-thirds of the metal. Thus the lesser area is far more effectively employed than the greater, the explanation being that the central portions of the transverse area of a pillar are less useful to resist bending than those at a greater distance, and therefore, when removed, they do not diminish the resisting power proportionally.

But if (the lengths being as before the same) the quantity of material or weight in the solid and the hollow pillar be equal, or, which is the same thing, if the transverse section of each be equal, then the hollow pillar is in every point of view the stronger.

151. The woodcut annexed will illustrate this last point: in it A*a* is the diameter of the transverse section of a solid pillar, and, being equal to half an inch, will represent, at a scale of one-eighth, a cylindrical pillar 4 inches in diameter; if at the point A, the extremity of the diameter A*a*, we erect an

indefinite perpendicular, we may, by a simple geometrical construction, obtain either the internal or external diameters of the annulus, equal to the area of the circle, by being given the other. Thus, if the internal diameter be given, lay out, upon the indefinite tangent, from A, a line, as $\overline{Ab}$, equal to it, and join the points a and b, then the line $\overline{a, b}$, will be the required external diameter. For the area of the annulus is equal to the difference of the squares of the diameters into 0.7854, that is, $(\overline{ab}^2 - \overline{bA}^2) \times 0.7854$, and from the right-angled triangle Aab this is equal to $\text{A}a^2 \times 0.7854$, which is the area of the semicircle. This annulus is represented in the second woodcut by the shaded space at B. The annulus at C, and also that at D, are equal in area to it, having as internal and external diameters the lines in the first woodcut, $\overline{\text{A}c}$ and $\overline{ac}$, and $\overline{\text{A}d}$ and $\overline{ad}$ respectively. Now the areas of these being equal, and the strengths, *cæteris paribus*, being as the 3.5 power of the internal subtracted from the 3.5 power of the external diameter, must increase continually; but the shell of the metal becoming thinner also, a practical limit is soon found by the difficulty of obtaining sound and true castings.





152. At the scale of $\frac{1}{8}$th the diameter of the solid pillar A will be 4 inches, and the external diameters of the hollow pillars, whose sections are B, C, D, are 5.5 inches, 8 inches, and 13 inches, respectively. As the areas of these are all equal to that of the circle A, we can calculate the internal diameters $\overline{\text{A}b}$, $\overline{\text{A}c}$, $\overline{\text{A}d}$, thus:—as

$$0.7854(5.5^2 - \overline{\text{A}b}^2) = 0.7854 \times 4^2,$$

we have

$$\overline{\text{A}b} = \sqrt{30.25 - 16} = 3.775 \text{ inches},$$

giving a thicknesss of 0.8625 inches for the ring B. In like manner,

$8^2 - \overline{\mathrm{A}c}^2 = 4^2$, giving $\mathrm{A}c = 6.928$ inches,

and 0.536 inches for the thickness of section C. And again, $\mathrm{A}d = 12.37$ inches, and the thickness of D equal 0.315 inches.

To compare the relative ultimate strengths of these four columns, all of the same transverse area, as depending on their different diameters, we will assume them to be of the length l, the same in each. And therefore we may omit from the expression $42.35 \times \frac{d^{3.5} - d_1^{3.5}}{l^{1.63}}$, as common to each, the quantities 42.35, and $l^{1.63}$. The relative bearing powers of each will therefore be

$$4.0^{3.5} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad = 128.0 \; . \; . \; 1.00.$$
$$5.5^{3.5} - 3.775^{3.5} = 390.2 - 104.5 = 285.7 \; . \; . \; 2.23.$$
$$8.0^{3.5} - 6.928^{3.5} = 1448.15 - 875.24 = 572.9 \; . \; . \; 4.50.$$
$$13.0^{3.5} - 12.370^{3.5} = 7922.8 - 6657.24 = 1265.6 \; . \; . \; 9.89.$$

The thickness of 0.315″ in a column 13 inches in diameter would, however, be unadvisable, and it is probable that an external diameter of 9 inches would be the greatest that this transverse section of metal, equal to 12.566 square inches, could be extended to, as a good sound casting. This would give an internal diameter of 8.062 inches, and a thickness of 0.47 inch, which may be taken as half an inch, and the relative number, to compare with $4^{3.5} = 128$, would be 699.2, giving an increase of strength in the ratio of 1 to 5.46, or five and a half times stronger than a solid pillar having the same quantity of metal.

153. The problem of calculating the proper dimensions of cast iron columns presents itself in practical construction chiefly in one of these two forms:—

Either it may be required to assign the requisite diameter, if solid, or the external and internal diameters, if hollow, being given the load to be supported, and the clear height of the pillar—under this head many examples are given in §§ 158 to 161.

Or, on the other hand, it may be required to calculate the load that a pillar will sustain, all whose dimensions are given, the coefficient of stability being duly applied in each case. The examples under this division of the subject will be found in §§ 161 to 172.

In the first case the problem is indeterminate when the pillars are to be hollow, though the data are sufficient when solid; and it therefore assumes different forms, according to the different conditions which must be added to give the question the necessary definiteness.

154. 1st. We may have the ratio of the length to the external diameter, as suppose 40 to 1, or 35 to 1, or some other, given, and thus it only remains to calculate the internal diameter; for, as the length is supposed to be among the data, the ratio above mentioned determines the external diameter, from which the internal is easily computed.

155. 2nd. It may be the condition that the internal diameter is to be some given fraction of the external, though this, it must be remarked, could never be applied generally; for it would give too great a thickness for the larger, and too small for the lesser diameters.

156. 3rd. We may, together with the clear height, and the load to be supported, have the thickness of the ring of metal given, to calculate the diameters.

157. 4th. The transverse area of the ring of the metal may be given, which is, in other words, to be given the weight of the column itself, since, as the length is given, the volume, which is the transverse area into the length, is known; and thus, from the weight of a cubic inch or cubic foot of cast iron, that of the pillar is readily obtained. This fourth head will enable us to compare the strength of a solid pillar with that of a hollow one of the same weight, or to compare two hollow cylinders of the same weight; or, again, to design a hollow cylinder which shall be twice, or any other proportion, as strong as another given column of the same weight.

158. In the first of these subdivisions of the 1st case, § 154, let the greatest probable load which can come upon the pillar to be designed be equal to 71.75 tons, and the "coefficient of safety" 7; also let the ratio of the height to diameter be 30 to 1, and the given length of the column equal to 29 feet, so that the resulting external diameter will be 0.966 feet, or 11½ inches, and this we will assume it to be, so that, with this added condition, the question is to calculate the internal diameter. Let z be the internal diameter, then

$$42.35 \times \frac{11.5^{3.5} - z^{3.5}}{29^{1.63}} = 7 \times 71.75 = 502.25 \text{ tons.}$$

$$\text{Hence } \log z = \log\left\{11.5^{3.5} - \frac{502.25 \times 29^{1.63}}{42.35}\right\} \div 3.5$$

$$= \log\{2288.14\} \div 3.5$$

$$= 3.35948 \div 3.5 = 0.959851 \text{ answering to } 9.116 \text{ inches,}$$

so that the thickness of the ring would be 1.2 inches.

The cubic content of this column, if of a uniform thickness, will be

$$(11.5^2 - 9.5^2)\,0.7854 \times (29 \text{ ft.} \times 12) = 11{,}479 \text{ cubic inches.}$$

It will be evidently much simpler, with the above diameters, to take the product of the sum and difference, that is, $21 \times 2 = 42$, rather than to square the numbers, and then subtract the lesser. The above volume, reduced to cubic feet, is equal to 6.65; and if we take the weight of cast iron at 5 cubic feet to the ton, it will weigh 1.33 tons, which, at the price of £8 10*s*. per ton, would make the cost of each pillar to be £11 6*s*. 8*d*.

The specific gravity of cast iron being taken at 7.2, and the weight of water at 62.3 lbs. per cubic foot, would give 448.56 lbs. per cubic foot as the weight of cast iron, and this multiplied by 5 gives 2242.8 lbs. only, about ⅛th per cent. too great.

To facilitate the calculations necessary in determining the dimensions of columns, there are given at the end of these examples Tables of the 3.5th and 1.63rd powers of numbers, extending as far as the greatest dimensions of diameter and length to be found in ordinary practice. In the Table of the lengths raised to the 1.63 power there is also added the logarithm of the constant 42.35, divided by the number on the same line raised to that index; this division occurs so frequently in the tentative process of determining the diameters of hollow columns to sustain a given load, that it diminishes the labour very much to be able to take it out at once from a Table. Thus we have in the Table of 1.63 powers, opposite 29 feet, and on the same line,

$$29^{1.63} \quad | \quad 241.94 \quad | \quad 1.2431457,$$

the logarithm being that of ($42.35 \div 241.94 =$) 0.1750435, a division which occurs frequently in the determination of the ultimate strength from given dimensions of the pillar. If, as in the example in page 136, we have to determine the dimensions from the load to be sustained, as one of the given quantities, then we find, on the contrary, $241.94 \div 42.35 = 5.71287$. This we immediately obtain by subtraction.

From, ... 0.0000000
Subtract log opposite $29^{1.63}$, $\bar{1}.2431457$

Log of quotient, ... 0.7568543

The whole subject requiring, as it does, the frequent raising of numbers to fractional indices, and extracting the same roots is the proper province of logarithmic calculation, to which also the formulæ are well adapted; but as many persons busily engaged in practice may not have their former facility in using it, a diagram has been given in fig. 35, in which are plotted on a horizontal line the diameters in inches, at a scale of one-third, all from a common zero; and, at the point answering to each diameter, a vertical is raised, equal to its 3.5th power, at a scale of 1000 to an inch; a curved line being drawn through all the points so found, we can by it obtain either the 3.5th power or 3.5th root of any number within the range of the extreme values of the horizontal and vertical lines in the figure. As the verticals would, at this scale, be inconveniently long with the larger diameters, the curve is dropped down 5000 after 12 inches diameter, and this number must be added to the readings of the verticals with diameters greater than that.

The use of the diagram, though not giving very precise results, is very simple. Thus, on the proper point on the horizontal, answering to the given external diameter, lay the ordinary scale of 10 truly perpendicular to it, and having its zero in the same line; then the point at which the curve appears to cut the edge gives the 3.5th power which is sought, each subdivision on the scale reading one hundred.

To find the 3.5th root of any number, lay off the given number on any vertical, by the same scale of 10, and through the point thus found draw a horizontal line to cut the curve; the perpendicular from this gives the number required on the horizontal. In the same way numbers may be found, the difference of whose 3.5th powers may be equal to a given quantity. The curve may also be very useful in suggesting numbers which have a given difference, and given difference of 3.5th powers, as will be explained in an example with such data.

It is evident that between any successive ordinates given in the Table, being generally only 0.1 apart, the curve is very nearly a right line, and therefore a near approximation of the 3.5th power may be obtained, if the diameter fall between any two successive numbers in the Table, by supposing the series of verticals to be for that interval an arithmetic series. Thus, to find the 3.5th power of 7.97, we have—

$$
\begin{array}{rcl}
8^{3.5} & = & 1448.15 \\
7.9^{3.5} & = & 1385.78 \\
\hline
\text{Differences,} \quad 0.1 \text{ and} & & 62.37
\end{array}
$$

Now, 0.10 being the difference of the nearest diameters, and 0.07 the excess of the given number above the next less in the Table, we must use the proportion,

$$10 : 7 :: 62.37 : 43.659,$$

in order to obtain a number to be added to 1385.78, for the approximate value of $7.97^{3.5}$. Thus,

$$1385.78 + 43.659 = 1429.439.$$

Now, log of $7.97 = 0.9014583$, and this multiplied by 3.5 gives 3.1551040, which answers to the number 1429.24, the approximate value being 0.199 greater, this exact result being always a little less than the approximate number, as the curve is convex towards the horizontal line, as is shown by the right lines joining the ends of the ordinates at the diameters, 4 and 5.

Again, to find the value of $7.77^{3.5}$, we have

$$\begin{array}{rcl} 7.80^{3.5} & = & 1325.35 \\ 7.75^{3.5} & = & 1295.85 \\ \hline \end{array}$$

$$\text{Differences,} \quad 0.05 \text{ and } \quad 29.50$$

and as the excess of the given number above the next less is 0.025, which is half the above difference, we simply add the half of 29.50 to $7.75^{3.5}$, that is, $1295.85 + 14.75$, giving 1310.60, the true value obtained by logarithms being 1310.445.

As another instance, the true value of $11.95^{3.5}$ being 5899.125, and the difference of the nearest numbers in the Table, which are $11.9^{3.5}$ and $12^{3.5}$, being 172.77, we must add the half of it to the lesser, namely, $5813.19 + 86.385 = 5899.575$, for the approximate value, which is 0.45 too great.

In a manner precisely similar we may obtain the value of any length, raised to its 1.63rd power, interpolating between the nearest numbers in the Table.

The "coefficient of safety" in designing cast iron pillars ranges from about 6 to 10. It is impossible to give any rule for its precise determination. The engineer must be guided by the greater or less probability of contingencies occurring, which would throw an unexpected increase of the estimated load upon the column: thus, if a weak foundation were to cause any one of an assemblage of pillars to settle down, however small a quantity, and that a continuous girder resting horizontally on the capital of this pillar was not sufficiently rigid, then its deflection would throw an undue proportion of the superincumbent weight on this particular column, and perhaps induce its fracture; at least, it would be much more severely strained than those not placed in the same circumstances, especially if it did not sink vertically, but received its pressure in a diagonal direction, in which case its strength is reduced one-third. On the other hand, if a strong and comparatively inflexible beam lay across the capitals of the pillars, then the load would not press with such severity on the deranged pillar, by continuing to follow its subsidence, but the surrounding ones would each have to bear some increase of their estimated load, and the structure above this horizontal beam would probably not suffer, by settlements or otherwise, from the defective pillar. The estimated load, referred to above, can be computed only by taking the weight of the structure itself, and everything that may be placed upon it, for half the distance between any particular support and all those that surround it, and supposing that it sustains this weight, and this only. Thus, if a warehouse-floor be supported by pillars, each row being 25 ft. apart, and those in any row 10 ft. asunder, then each pillar must be taken as loaded with the flooring, and all upon it, for a rectangular area, 12 ft. 6 in. on each side in the direction of the several rows, and 5 ft. on each side between pillar and pillar, that is, 25 ft. by 10 ft.

The proportions of the height to the diameter in the ancient Classic Orders have not been followed in the designs of cast iron pillars. The shaft of the Tuscan column is about six diameters of the base in height, that of the Doric about seven, of the Ionic about eight, and the Corinthian about nine diameters. The diminution of the diameter, from the base upwards to the capital, is also widely different from that used in cast iron pillars. The upper diameter in the Classic columns is about $\frac{5}{6}$th of that at the base, which gives a taper of from 1 in 72 to 1 in 108; but in pillars of cast iron it may be observed to be from 1 in 150 to 1 in 350. If the upper diameter of a column is made less than that at the base by an eighth of an inch for every foot of height, it will give a uniform taper of 1 in 192.

In the Albert Docks, at Liverpool, and at St. Catherine's Docks, London, however, we find cast iron pillars, nearly of the Grecian-Doric proportions, supporting warehouses many stories in height, with fire-proof floors, loaded with immense weights of merchandise. These may be considered as sustaining, probably, the greatest loads to be found in practical construction, and seem therefore well suited to their position, the height being but small, and the peculiar object of the building requiring the interruptions to the clear space to be as few as possible.

Returning to the consideration of the first subdivision, we thus perceive that, with this added condition, namely, the ratio of the length to the diameter, the problem of calculating the diameters always resolves itself into that of the determination of the requisite internal diameter, which we may call z, the external being always x.

As another example of this subdivision, let the required height of a pillar be 40 ft. 10 inches, and the ratio of the length to the diameter be 35 to 1—the utmost weight to be sustained by it, 60 tons, and the coefficient of safety 7. The external diameter is at once found by the proportion,

$$35 : 1 :: 490 : 14 = \text{mean external diameter};$$

so that

$$42.35 \times \frac{14^{3.5} - z^{3.5}}{40.83^{1.63}} = 7 \times 60 = 420 \text{ tons};$$

and as, from the Table of 3.5th and 1.63rd powers,

$$14.^{3.5} = 10{,}267.34, \text{ and } 40.83^{1.63} = 422.57,$$

we have

$$10267.34 - z^{3.5} = 4190.77.$$

Transposing

$$10267.34 - 4190.77 = 6076.6 = z^{3.5},$$

and therefore,

$$z = \sqrt[3.5]{6076.6} = 12.016 \text{ inches}.$$

The log of 6076.6 is 3.7836606, and this divided by 3.5 gives 1.081046, which answers to the number 12.016 inches, as above; so that this proportion of length to diameter will, with the other conditions, result in a thickness of 1 inch, which is the least, probably, that could be adopted in making such a casting.

If the column is intended to have a taper, the diameter at the base being greater than that at the top, then the ratio of the length to the diameter is always in these examples taken to be that of the length to the external diameter at the middle height: thus, in the last example, let the slope or taper of the pillar be $\frac{1}{350}$, this will make the diameter at the top 1.4 inches less, and that at the bottom also 1.4 inches greater than the mean of 14 inches, or 12.6 inches and 15.4 inches respectively.

In all the examples on calculating the diameters of cast iron pillars, it is supposed to be those at the middle of the height that are taken, if the column be tapering in form, from which those at the top and bottom are readily determined, by applying to them the given rate of the diminution.

159. In the second subdivision, § 155, of the first case, in which the added condition is that the internal diameter has some given ratio to the external, as an example, let this assigned proportion be $\frac{4}{5}$ths, and the requisite length of the column 30 feet, its estimated load being 60 tons, and "coefficient of safety" taken to be 8, so that the ultimate load is 480 tons. Then, x being the external diameter, we may express the internal z in terms of x, so that

$$42.35 \frac{x^{3.5} - (\frac{4}{5}x)^{3.5}}{30^{1.63}} = 480 \text{ tons.}$$

(The numerator above is equal to $x^{3.5}[1 - (\frac{4}{5})^{3.5}]$, and as $(\frac{4}{5})^{3.5} = \frac{128}{279.5} = 0.458$, and $1 - 0.458 = 0.542$, we have, finally, the numerator expressed by $x^{3.5} \times 0.542$.)

Reducing the above equation, and taking the logarithm, we have

$$\log x = \log \left\{\frac{480 \times 30^{1.63}}{42.35 \times 0.542}\right\} \div 3.5.$$

Opposite 30 in the Table of 1.63 powers, we find 255.69. Hence,

$$\begin{array}{rl} \text{add } \log 480 & = 2.6812412 \\ \text{to } \log 255.69 & = 2.4077137 \\ \hline & \ 5.0889549 \\ \text{subtract } (\log 42.35 + \log 0.542) & = 1.3608527 \\ \hline & \ 3.7281022 \end{array}$$

which, divided by 3.5, gives 1.065172, answering to the number 11.619 inches, for the external diameter; and as the internal is to be $\frac{4}{5}$ths of this, it will be equal to 9.30 inches, giving a thickness of the ring of metal of 1.16 inches.

If, as another example, a column be required to sustain 9 tons, and the coefficient of stability be taken at 6, the necessary height being 19 ft. 8¼ inches, calculate the external diameter, when the internal is to be $\frac{4}{5}$ths of it.

From the above data we have

$$42.35 \times \frac{x^{3.5} - (\frac{4}{5}x)^{3.5}}{19.686^{1.63}} = 54 \text{ tons};$$

so that

$$x^{3.5} = \frac{54 \times 128.67}{22.95} = 302.75,$$

and log 302.75 = 2.4810842, which, divided by 3.5, gives 0.709, answering to 5.117 inches for the exterior diameter; and consequently the interior is 4.0934, that is, very nearly $5\frac{1}{8}$ inches and $4\frac{1}{10}$ inches respectively. If this thickness, which is little more than $\frac{1}{2}$ an inch, be deemed, as it probably would be, too small for sound castings of that length, then it must be increased accordingly, the experience of the foundry deciding on the least suitable dimension; the column, so altered, would, however, be wasteful of material. Or the design of the structure itself may, with reference to the pillars, be altered, so that there will be fewer columns, each of them, therefore, carrying a greater load; their necessarily increased lateral dimensions will, if the number of columns be sufficiently reduced, give a safe practical thickness.

The above proportion of $\frac{4}{5}$ths gives a thickness of ring of $\frac{1}{10}$th of x; and, as already mentioned, this, or any other such rule for the relation of the diameters, would give too great a thickness with large diameters, and too small with lesser. Thus, 4 ft. external diameter gives, at the proportion of $\frac{4}{5}$ths for the internal diameter, a thickness of ring of 4.8 inches; and for a column 4 inches in diameter, a thickness of 0.4 inch, both inadmissible in practice from opposite defects: no example can be found of such a thickness as 4.8 inches; and most iron-founders would give the small pillar about 0.75 inch thickness, even if it made it much stronger than required.

It is sometimes stated that the thickness of the metal should be about $\frac{1}{12}$th of the external diameter; this rule would give an internal diameter $\frac{5}{6}$ths of the external. Whatever thickness of metal calculation may point out as required in a column, founders will always be anxious to avoid risking unsoundness by having it too thin.

160. Referring now to § 156, the third subdivision, let the condition added to those of a given height of the column and given load be the thickness of the ring of metal.

Thus, for example, let the estimated load upon a pillar be 40 tons, and the coefficient of stability be assumed equal to 6, the height of the pillar being 24 ft., and the thickness of the ring of metal 1 inch: calculate the requisite internal and external diameters.

From these data we shall have the following equation:—

$$x^{3.5} - (x - 2t)^{3.5} = n;$$

(or, in words, given the difference of two numbers, and the difference of their 3.5th powers, to determine the numbers.)

In which equation x is the external diameter, and t the given thickness of metal, so that $x - 2t$ represents the internal diameter, and n is equal to the given total load (that is, the estimated load, into the coefficient of stability), multiplied by the length raised to the 1.63 power, and the product divided by the constant, 42.35. Or, with the numerical data above, since $24^{1.63} = 177.723$,

$$x^{3.5} - (x - \overline{2 \times 1})^{3.5} = \frac{240 \times 177.72}{42.35} = 1007.2.$$

The known methods of solving such equations would be intricate and laborious, and to a great extent as tentative in the process as that by the use of the table of numbers raised to the 3.5th power, given at the end of these examples.

As we always have given, by the data under this, the third subdivision, both the difference of the diameters and the difference of their 3.5th powers, we may, by an inspection of the Table, discover numbers having very nearly this given difference, and difference of 3.5th powers, being guided in the search by a knowledge of the major and minor limits of the external diameter, obtained by applying to the given height some ratio intermediate between the greatest and least ratios of length to diameter usual in practice; but even this abbreviation of the search would leave it still too wide, and renders some calculation necessary.

We may proceed thus:—let us take 50 to 1, and 20 to 1, as the limits of the ratio of the length to the diameter adopted in practice, the mean being 35 to 1. With these ratios, and the given length of 24 ft., we have at once the following values of the external and internal diameters, namely—

(*a*) 50 to 1 . . 5.76 inches and 3.76 inches.
(*b*) 35 to 1 . . 8.23 „ 6.23 „
(*c*) 20 to 1 . . 14.4 „ 12.4 „

Now, the diameters, 5.75 and 3.75, being the nearest in the Table of 3.5th powers to those of 5.76 and 3.76, as deduced above, we may, in the first approximations, assume these to be the least diameters admissible with a height of 24 feet; and in like manner we may take 8.25 and 6.25 for 8.23 and 6.23. The numbers 14.4 and 12.4 are actually found in Table. We have, therefore, the following results:—

$$(a) \ldots 42.35 \times \frac{455.87 - 102.12}{177.72} = 84.3 \text{ tons.}$$

$$(b) \ldots 42.35 \times \frac{1612.83 - 610.35}{177.72} = 238.8 \text{ tons.}$$

$$(c) \ldots 42.35 \times \frac{11331.31 - 6714.1}{177.72} = 1101 \text{ tons.}$$

The result obtained in (*b*), namely, 238.8 tons, is so nearly equal to 240 tons, the load to be supported by the intended column, that we may take the external and internal diameters to be 8.25 and 6.25 inches, respectively.

Again, as another example, let the estimated load to be supported be equal to 140 tons, and the length of the pillar, as above, be 24 ft., the coefficient of stability being also equal to 6 in this case, so that the total load to be provided for will be 840 tons.

If we try the diameters, 14 and 12 inches, we find the resulting strength to be 1020.2 tons, which is too great. Again, 13.5 and 11.5 inches give 925.2 tons, also too great. But 13 and 11 inches give 835.75 tons, showing that these diameters are very nearly equal to those we are seeking.

The tentative process is greatly facilitated by using the logarithm opposite 24 ft., in the Table of 1.63 powers, as already explained. Thus, from the Table of 3.5th powers, we find at once that $13^{3.5} - 11^{3.5} = 3507$. Therefore,

To log opposite $24^{1.63}$, which is $\bar{1}.3771171$
Add log of 3507, ... 3.5449358

Product 835.7, 2.9220529

the ultimate strength in tons, as mentioned above. And, in like manner, we may obtain the strength of any other external and internal diameters of a pillar of given length.

The diagram, fig. 35, may be used to suggest approximately the diameters of which to make trial: mark off on any two conterminous edges of a piece of drawing paper, having the sides about the angle truly perpendicular to each other, the given difference of diameters—measured from the right angle—

on one edge, and the given difference of 3.5th powers on the other, each by its proper scale; then, keeping each of them parallel to its proper line on the diagram, that is, the difference of the diameters parallel to the horizontal line, and that of the 3.5th powers to the vertical, move the paper about until the points, marked off as above directed, are each found simultaneously upon the curve; then perpendiculars let fall from these points on the curve to the horizontal will give the required diameters very nearly.

This result may also be attained by constructing a diagram curve, obtained by plotting to scale a few of the first results of the tentative calculations, in which we may use such diameters as are found in the Table—taking some which give a greater, and some a less, ultimate strength than the proposed load to be supported. Fig. 36 exhibits such a curve with the ordinates and abscissæ drawn to scale. In it the curve is given through a very wide range for the sake of illustration, but for any particular result a very few points on each side of that which is sought are sufficient.

The vertical scale represents tons, at 200 tons to an inch, having its zero in the horizontal line, and therefore each single ton can be read off by the dividers on the small diagonal lines on the ordinary Gunter's scale.

The horizontal scale represents inches, at 3 inches to an inch, or the 30 scale, by which the value of the diameters can be obtained to the second place of decimals of an inch. This particular curve, which is only adapted for lengths of pillar of 24 ft., and a thickness of metal of 1 inch, is thus constructed:—at any point on the horizontal line, as 8 inches, for an external diameter, raise a vertical equal to the tons in the ultimate strength, obtained by the following method:—

Take out the 3.5th powers of 8 and 6 from the Table—the internal diameter being 2 inches less than the external, with the given thickness of 1 inch.

Then, $8^{3.5} - 6^{3.5}$ being equal to 919.06,

we have

$$\begin{array}{lr} \text{Log opposite } 24^{1.63} = & \bar{1}.3771171 \\ \text{Add log } 919.06 = & 2.9633439 \\ \hline \text{Product, } 219.01, & 2.3404610 \end{array}$$

which is the vertical plotted at the point on the horizontal answering to 8 inches external diameter.

In the same manner we have

$$6^{3.5} - 4^{3.5} = 401.09;$$

and hence

To . . . . . . $\bar{1}.3771171$
Add log of $401.09 = 2.6032418$

Product, 95.58, 1.9803589

Being the vertical at 6 inches diameter on the horizontal.

Again, $4^{3.5} - 2^{3.5} = 116.686$.

To . . . . . . $\bar{1}.3771171$
Add log $116.686 = 2.0670336$

Product 27.81, 1.4441507

And in like manner all the other ordinates may be obtained, through the upper extremities of which the curve is drawn, by a flexible ruler, or suitable French curve.

It is evident that the first ordinate must, in this case, be at the point answering to a diameter of 2 inches; for, as the thickness of the ring of metal is, in this example, one inch, the internal diameter will become equal to zero when the external is equal to twice that given thickness. Its value is thus found :—

To the log opposite $24^{1.63}$ in Table, viz., $\bar{1}.3771171$
Add log of $2^{3.5} = 11.314 = 1.0536162$

0.4307333

answering to the number 2.696, which is the ordinate at the absciss equal to 2 inches.

Having thus obtained a curve representing the ultimate strengths of columns 24 ft. high, with different diameters, and having a thickness of one inch of metal, we may by it, in a reverse manner, determine the diameters of a column intended to sustain a given load.

For example, let us suppose this load to be 750 tons. Lay off the length answering to the given load on the vertical scale of tons, obtaining the proper length from a diagonal scale reading the hundredth part of half an inch: through the point so found draw a horizontal line to cut the curve, and from this point on the curve let fall a perpendicular cutting the horizontal scale of inches at a distance upon it which gives the required external diameter of the column, the internal being found by deducting twice the given thickness of metal from it. It will be seen that, in this example, the vertical cuts the horizontal line at a point a little short of 12.5. Let us try 12.49, and by actual calculation find the strength of the column 24 ft. high, and having an external diameter equal to 12.49 inches, and an internal of 10.49 inches; in other words, with a thickness of one inch of metal.

Then, $12.49^{3.5} - 10.49^{3.5} = 6886.02 - 3738.64 = 3147.38$.

$$\begin{array}{rr} \text{To log opposite } 24^{1.63} \text{ in Table, viz.,} & \bar{1}.3771171 \\ \text{Add log } 3147.38 = & 3.4979436 \\ \hline & 2.8759607 \end{array}$$

answering to 749.999, and showing that we may rely on the curve for very near results.

161. If, in addition to the height of the column, and the ultimate load to be supported, we have, as stated in the fourth subdivision of the problem, § 157, the transverse area of the ring of metal, as the added condition given with those above named, the following equations obtain, in which, as before, the external diameter is represented by x, and the internal by z.

$$x^{3.5} - z^{3.5} = \text{Ultimate Load} \times \frac{l^{1.63}}{42.35}.$$

In which the Ultimate Load represents the estimated load multiplied by the coefficient of safety, and l, the known length of the pillar.

And, secondly,

$$x^2 - z^2 = \text{Given transverse area} \div 0.7854.$$

Or, in words, given the difference of the squares of two numbers, and the difference of their 3.5th powers, to determine the numbers—a problem which by no means surpasses the resources of algebra, but the process would be too intricate for the practical determination of the diameters of a pillar, and in the end, moreover, involve methods of proceeding as completely tentative as those about to be given.

For instance, selecting a certain external diameter, from it we easily deduce the internal diameter answering to the condition that the given transverse area of the ring of metal may be contained between them; and having thus all the dimensions of the column, we can calculate its ultimate strength by the aid of the Table of 3.5th powers of numbers, taking out the nearest values of the diameters to be found in it for the first approximation.

We may, however, very greatly limit the range of the numbers from which to select the values of the external diameters, by a knowledge of certain rules or customs derived from long experience, which obtain in practice. Thus, in the first place, we may observe in good examples a major and a minor limit to the thickness of the ring of metal in a hollow pillar, in the very largest of which, of late years, the thickness does not exceed, if indeed it come up to, two inches; and, on the other hand, the smallest columns are never much less than half an inch,—it being advisable, and in the end more economical, to use a greater quantity of metal than is absolutely necessary, rather than risk any unsoundness in the casting: a defective casting must, if it be perceived, be broken up and re-melted at a considerable cost, or, if not perceived, may endanger the structure in which it is applied. And if this rule as to thickness be not sufficient, we may also abridge the labour of this tentative determination of the unknown transverse dimensions of the required pillar, by applying, as in the third subdivision, § 160, some ratio of height to mean external diameter, falling within the usual limits of this proportion, which may be taken to be, as already intimated, from about 50 to 1 to about 20 to 1.

First, then, let us investigate the method of determining the external and internal diameters of a hollow column, when we are given the transverse area of the annulus, and have selected its thickness, which is always half the difference of the diameters.

We have, therefore,

$$0.7854\,(x^2 - z^2) = \text{given area};$$

so that

$$x^2 - z^2 = \text{given area} \div 0.7854;$$

and as $x^2 - z^2 = \overline{x + z} \times \overline{x - z}$, and $x - z$ is equal to twice the assumed thickness, we have

$$\text{sum of diameters} = (\text{given area} \div 0.7854) \div (2 \times \text{given thickness}).$$

Again, as half the sum, plus or minus half the difference of two unequal numbers, is equal to the greater or the less, according as we take the former or the latter sign, we thus obtain the external and internal diameters, which correspond to the assumed thickness.

As an example—let us suppose a certain structure requires a column 29 ft. high, and that the estimated load it has to support, multiplied by the coefficient of safety, gives an ultimate load of 480 tons, the supposed transverse area to be employed amounting to 30.2 square inches,

We have $x^2 - z^2 = 30.2 \div 0.7854 = 38.4517$ square inches.

And, bearing in mind what has been stated as to the prudent or customary thickness of the metal, let us assume a thickness, first of 0.75 inch, and again of 1.25 inch, giving a difference of diameters of 1.5 and 2.5 inches, respectively. The sum of the external and internal diameters in these two cases will be $38.4517 \div 1.5 = 25.6345$ inches, and $38.4517 \div 2.5 = 15.3807$ inches, respectively.

And therefore we shall have

$$\frac{25.6345}{2} \pm \frac{1.5}{2} = 13.57 \text{ and } 12.07 \text{ inches,}$$

for the external and internal diameters, with the first value of the thickness.

Again,

$$\frac{15.3807}{2} \pm \frac{2.5}{2} = 8.94 \text{ and } 6.44 \text{ inches,}$$

for those of supposed greater thickness of 1.25 inches.

The transverse areas of each of these pillars will be equal to that with which it is proposed to construct the column, namely, 30.2 square inches, but their ultimate strengths will be very different. We will have, for the thickness of 0.75 inch, the equation,

$$42.35 \times \frac{13.57^{3.5} - 12.07^{3.5}}{241.94} = \text{Ultimate strength.}$$

The value of $29^{1.63}$ being taken from the Table of numbers raised to the index 1.63, opposite to which we also find the log $\bar{1}.2431457$, deduced below,

Now, as an approximation, we may take from the Table of 3.5th powers the numbers answering to 13.60 and 12.10, which are the nearest to those above determined, namely, 9276.54 and 6162.40, the difference between them being 3114.14.

Hence, $\log 42.35 - \log 241.94 + \log 3114.14 = \log$ ultimate strength.

$$\begin{aligned} \text{From } \log 42.35 &= 1.6268534 \\ \text{subtract } \log 241.94 &= 2.3837077 \\ \hline \text{log of quotient, } &\bar{1}.2431457 \end{aligned}$$

Now, as the length is given, we may use this resulting log in all our trials for suitable diameters, without any further trouble in each particular case, but to find the log of the difference of the 3.5th powers of the diameters.

Hence

$$\begin{array}{rl} \text{To} \; . \; . \; . \; . \; . & \bar{1}.2431457 \\ \text{Add log } 3114.14 = & \underline{3.4933382} \\ & 2.7364839 \end{array}$$

answering to the number 545.1, which is therefore the ultimate strength in tons, with these diameters of the column, being 65.1 tons more than is required.

If we calculate for the exact diameters resulting from the thickness 0.75, namely, 13.57, and 12.07, we have

$$\begin{array}{lr} 3.5 \times \log 13.57 = 3.9640293, \text{ which gives} & 9205.11 \\ 3.5 \times \log 12.07 = 3.7859755, \quad ,, & \underline{6109.12} \\ \text{Difference of } 3.5\text{th powers of diameters,} & 3095.99 \end{array}$$

hence,

$$\begin{array}{rl} \text{To log opposite } 29^{1.63} = & \bar{1}.2431457 \\ \text{Add log } 3095.99 = & \underline{3.4907995} \\ \text{log of quotient,} & 2.7339452 \end{array}$$

answering to 541.932, showing the utility of the Table, the use of the nearest numbers in which gave a result only differing from the former by about 3 tons.

It remains next to calculate the ultimate strength for the thickness of 1.25 inches, with the same length, load, and transverse area. The external and internal diameters, we have seen, must now be 8.94 and 6.44 inches. The nearest to these in the Table of 3.5th powers will be 9 and 6.50, which, when raised to that index are

$$\begin{array}{rl} 9^{3.5} & = 2187.00 \\ 6.50^{3.5} = & \underline{700.16} \\ \text{Difference,} & 1486.84 \end{array}$$

and, as before,

$$\begin{array}{rl} \text{To log opposite } 29^{1.63}, & \bar{1}.2431457 \\ \text{Add log } 1486.84 = & \underline{3.1722642} \\ \text{log quotient,} & 2.4154099 \end{array}$$

answering to 260.26 tons, which is too small by 219.74 tons.

It is evident that these ultimate strengths of the pillar are proportional to the difference of the 3.5th powers of the diameters, 542 tons having the same ratio to 260 that 3096 has to 1487.

To obtain the exact diameters, we may either proceed tentatively, by making trial of thicknesses intermediate between 0.75 and 1.25 inches, which we see can be effected without much labour. Or we may form a diagram, showing by a curve, as in the last section, the increase of strength with the increase of the external diameter, when the transverse area is given and constant.

Let the horizontal line, fig. 37, represent the external diameters in inches, at the same scale as figs. 35, 36, the verticals also, as in fig. 36, representing tons, at 200 tons to an inch. With a given transverse area of the ring, as 30.2 square inches, it is evident that the first vertical must be at the point on the horizontal which marks the diameter of a solid pillar, having its transverse area equal to the given annular area; for, as we gradually decrease the external diameter, the internal must also diminish, and finally become equal to zero, and at the same time, the assumed thickness of the ring must become equal to half the external diameter. With this final value we have, in the present example, $0.7854 \times x^2 = 30.2$ sq. inches; and consequently, $x = \sqrt{38{\cdot}4517} = 6.2$ inches.

The position of the first ordinate being thus obtained, its length is determined by calculating the ultimate strength of a solid pillar, 6.2 inches in diameter, and 29 ft. long. In the Table we have $6.2^{3.5} = 593{\cdot}43$,

$$\begin{aligned} \text{and to log opposite } 29^{1.63} \text{ ft., } & \bar{1}.2431457 \\ \text{add log } 593{\cdot}43 = {} & \underline{2.7733695} \\ \text{log of product, } & 2.0165152 \end{aligned}$$

giving 103.88 tons, which, taken from the diagonal scale, is plotted in fig. 37 at 6.2 inches on the horizontal line.

It is evidently not necessary to plot the whole curve in order to obtain the diameters for any particular strength of pillar: having obtained, as above, external diameters greater and less than that required, we may confine the calculation of the ordinates to those that lie between those two points on the horizontal line.

But in order to exhibit the curve for this example for a considerable portion of its length, the following ordinates, for ultimate strength in tons, have been calculated:—

Solid pillar, 6.2 inches diameter, 29 ft. long,				103.88
Hollow do.,	8 inches and	5.0545	inches, . .	202.67
,,	10 ,,	7.845	,,	316.82
,,	12 ,,	10.27	,,	440.17
,,	13.57 ,,	12.07	,,	541.93

Through the points thus found a curve was carefully drawn, and 480 having been laid off on the vertical scale of tons, and a parallel drawn through the point so found to cut the curve; and, again, from this point on the curve a perpendicular being let fall upon the horizontal line of inches, it was found to intersect at 12.62, giving the external diameter of the proposed column: the internal is found by the following calculation:—

$$12.62^2 - z^2 = 38.4517 \text{ square inches;}$$

consequently,

$$z^2 = \sqrt{159.26 - 38.45} = 10.99 \text{ inches.}$$

We may check the accuracy of this method of obtaining by diagram the external diameter, by calculating what the ultimate strength of this pillar would be, and comparing it with the given load of 480 tons.

Thus,

$$12.62^{3.5} = 7140.16$$
$$10.99^{3.5} = 4400.40$$
$$\text{Difference} = 2739.76$$

Hence

$$\text{To log opposite } 29^{1.63},\ \bar{1}.2431457$$
$$\text{Add log } 2739.76 = 3.4377125$$
$$\text{log of product, } 2.6808582$$

answering to 479.576.

This was the result of the very first trial of the curve, which cannot be expected in all cases to give such close approximations.

The pillar will, therefore, have a thickness of 0.815 inch, and a ratio of length to mean external diameter of 27.6 to 1. This thickness is less than $\frac{1}{12}$th, and it would be prudent to increase it to 1 inch at least; the external and internal diameters will therefore be 12.62 and 10.62, and the area increased from 30.2 to 36.5 sq. inches, the consequent increase in the ultimate strength being from 480 to 566.6 tons. If this last increase had been in the ratio of the increase of area, it would have been 584 tons.

It will be evident that, as the data of this fourth subdivision are sufficient, any other conditions added to them, and which may not be consistent with them, such as the thickness of metal, &c., will give the problem the character of determining the question, Can a pillar with a given transverse area be constructed with such and such dimensions, or of such a weight or cost, which depends upon the weight?

The area being given, evidently determines the weight, which may be calculated thus:—since 5 cubic feet of cast iron weigh a ton, a column, solid or hollow, 5 feet in height, and 1 square foot in section, will weigh one ton; and the weight of the same, if, as in the present example, 29 ft. high, will be

$$5 : 29 :: 1 : \frac{29}{5} = 5.8 \text{ tons.}$$

But the transverse area is only 36.5 sq. inches, instead of 1 square foot; therefore,

$$144 : 36.5 :: 5.8 : 1.47 \text{ tons,}$$

neglecting the weight of the square flat base and top or capital. And at £9 10*s*. per ton, the cost of each column will be £14 very nearly.

We may now consider the other tentative solution of the questions which arise under this subdivision, namely, by the aid of an assumed ratio of length to diameter, keeping in view good precedents derived from structures which have long fulfilled their intended purpose, and all the other practical limits already mentioned, which may be observed to obtain.

As another example, let us suppose a column 25 ft. high, intended to support a load which, multiplied by the "coefficient of safety," duly decided upon by reference to the special circumstances of the case, amounts in one part of the structure to 600 tons, and in another to 380; the transverse area of the ring of metal being 29.5 square inches in the section taken at half the height of the pillar, when it is constructed of a tapering form. Calculate, in each of these cases, the external and internal diameters of the pillars. We may, in conclusion, bring both the resulting thickness of the ring of metal and the ratio of length to lateral dimension to the test of acknowledged ordinary rules of practice, and, if needed, modify the given area by either enlarging or diminishing its amount, as the result of the investigation may point out.

If we apply to the given length of pillar, which is 25 ft., the extreme ratios of height to diameter, of 50 to 1, and 20 to 1, we shall obtain for the first ratio an external diameter of 6 inches, and for the second of 15 inches. The former, even if it were not considered too slender, would be impossible, with an area of ring of 29.5 sq. inches; for the transverse area of a solid pillar, 6 inches in diameter, is only 28.274 square inches. The diameter of the circle equal in area to 29.5 square inches is 6.128 inches.

The external diameter of 15 inches would certainly, in most situations, be considered to have the opposite defect to that of being too slender; and tested also by the resulting thickness of the metal, the transverse area of the ring being 29.5 square inches, will be found unsuitable from a deficiency in this point of view.

The internal diameter z is thus found from the equation,

$$0.7854\,(15^2 - z^2) = 29.5.$$

Solving for z^2 we have

$$15^2 - \frac{29.5}{0.7854} = z^2;$$

and, therefore,

$$z = \pm\sqrt{225 - 37.56} = 13.69 \text{ inches};$$

so that the difference of the diameters will be $15 - 13.69 = 1.31$, and the thickness of the ring 0.66 inch, which is much too small, being less than $\frac{1}{12}$th of the outside diameter, or even the very usual thickness of 1 inch.

For the ultimate strength we have, since $25^{1.63} = 189.95$,

$$42.35 \times \frac{15^{3.5} - 13.69^{3.5}}{189.95} = \text{Ultimate load in tons.}$$

Opposite $25^{1.63}$ in the Table we find a logarithm obtained as follows:—

From log 42.35 = 1.6268534
Subtract log 189.95 = 2.2786393

Constant log for this given length = $\bar{1}.3482141$
Add log ($15^{3.5} - 13.69^{3.5}$ =) 3578.12 = 3.5536549

log of product, 2.9018690

answering to 797.75 tons, indicating a higher strength than required, namely, 600 tons.

If, again, we assume the ratio of length to diameter to be 25 to 1, we shall have an external diameter equal to 12 inches, which, with the given area of the transverse section of the metal, determines z, the internal. Thus,

$$0.7854\,(12^2 - z^2) = 29.5 \text{ square inches.}$$

Solving for z we have

$$z = \pm\sqrt{144 - 37.56} = 10.317,$$

which would give a thickness of 0.841 inches, being rather less than good practical examples would warrant.

For its ultimate strength we have, since

$$12^{3.5} - 10.317^{3.5} = 5985.96 - 3527.26 = 2458.7,$$

as before,

$$\begin{array}{rr} \log\,(42.35 \div 189.95) = & \bar{1}.3482141 \\ \text{Add } \log\, 2458.7 = & 3.3907055 \\ \hline \text{log of product,} & 2.7389196 \end{array}$$

answering to 548.175 tons, which is below that which it is sought to obtain.

Intermediate between 15 and 12 let us, in order to draw a curve as in last section, make trial of 13.5 inches, which is a ratio of nearly 22 to 1; for the corresponding internal diameter we have, as before,

$$z = \pm\sqrt{182.25 - 37.56} = 12.02 \text{ inches};$$

the resulting thickness will therefore be 0.74 inch, the difference of diameters being 13.5 − 12.02 inches, and for the ultimate strength,

$$\begin{array}{rr} \text{To log} & \bar{1}.3482141 \\ \text{Add } \log\,(13.5^{3.5} - 12.02^{3.5} =)\ 3019.04 = & 3.4798689 \\ \hline & 2.8280830 \end{array}$$

answering to 673.1 tons.

With a ratio of very nearly 29 to 1, we will have external and internal diameters of 10.39 and 8.39, respectively, giving a thickness of one inch, and the difference of their 3.5th powers being 1903.71; so that

$$\begin{array}{r} \text{To log } \bar{1}.3482141 \\ \text{Add log } 1903.71 = 3.2796008 \\ \hline 2.6278149 \end{array}$$

answering to 424.35 tons.

We may with these four results, namely, 797.75, 673.10, 548.17, and 424.83 tons, and their several corresponding external diameters, construct, as in the last section, a curve, the external diameters being laid off as abscissæ on a horizontal line, Fig. 38, from a common zero, and the ultimate strengths as vertical ordinates, the respective scales being, for the former, 3 inches to an inch, and the latter, 200 tons to an inch. By the aid of this curve we may obtain the lateral dimensions of the pillar from its given ultimate load, or, conversely, from its external diameter obtain the corresponding strength,—each different height and transverse area of a column requiring an appropriate curve. It is only necessary to obtain a few points on each side of the required dimension, and draw that small portion of the curve that they give to obtain the required result, which therefore entails but a small amount of labour. A very considerable length of it is given in Fig. 38, from its first point to about 900 tons of ultimate strength. The first point is thus found:—we must calculate the vertical ordinate or ultimate strength of a solid pillar, having an area equal to that of the given annulus, and draw this vertical at the point on the horizontal answering to the diameter of this solid pillar, which diameter is evidently the minor limit of external diameters, with a given area of the ring of metal; for, with it, the internal has necessarily become equal to zero. This dimension is thus obtained:—since $0.7854 \times d^2 = 29.5$, we have $d = \pm \sqrt{37.56} = 6.1286$; and for the ultimate strength of this pillar, 25 ft. high, we have

$$\begin{array}{r} \text{log opposite } 25^{1.63} = \bar{1}.3482141 \\ \text{Add log } (6.1286^{3.5}) = 2.7557645 \\ \hline \text{log of product} = 2.1039786 \end{array}$$

answering to 127.05 tons, which is the vertical plotted in fig. 38, at 6.1286. Those at 8 and 7 inches external diameters were found by calculation to be 254.14 and 186.46 tons, respectively, and plotted at these points at the proper scale.

The application of the curve is very simple. Thus, through the point on the vertical scale corresponding to 380 tons, Fig. 38, draw a line parallel to the line of diameters; this gives us a point on the curve, from which, if we let fall a perpendicular on the horizontal line, its intersection will be found to be at 9.8, which is, therefore, the external diameter in inches. By the method of calculation already detailed, we find the internal corresponding to it, so that the area of the annulus between them may be equal to that proposed, namely, 29.5 square inches, is 7.647 inches, and therefore the thickness of the ring of the metal will be 1.08 inches. Checking these results of the diagram by actual calculation, we find that with the above diameters of 9.80 and 7.6472, the ultimate strength is 381.2 tons, which is a sufficiently near result.

In like manner, drawing a horizontal line through the point on the vertical corresponding to 600, we find the perpendicular from it cuts the line of diameters at 12.65 inches; this is the external diameter, and from it the internal is deduced by the expression $z = \pm\sqrt{160.0225 - 37.56} = 11.066$.

Checking, as before, the accuracy of the method of solving the question by diagram, we calculate the strength of a hollow pillar of the length 25 ft.; and with 12.65 and 11.066 inches, as given by the diagram, for the external and internal diameters, we find then, first, that $12.65^{3.5} - 11.066^{3.5} = 2691.94$. Therefore,

$$\begin{array}{rl} \text{To log opposite } 25^{1.63}\text{, which is} & \bar{1}.3482141 \\ \text{Add log } 2691.94 = & 3.4300653 \\ \hline \text{Product, } 600.177\text{, answering to} & 2.7782794 \end{array}$$

But the thickness of the metal with the above lateral dimensions would only be 0.792 inches; and if to obtain a greater thickness, with this transverse area, we reduced the external diameter, the strength would then also be reduced, so that if that thickness be deemed too small, as it probably would be, we must, to acquire the proposed strength, add to the area.

We may take the assistance of another principle in shortening the process of tentative determination of the lateral dimensions of hollow pillars, namely this; the ultimate strength of a hollow pillar of any length, whose external diameter is double that of a solid pillar of the same transverse area and length, is 4.5 times greater than that of the solid pillar. We will take solid pillars of 3, 5, and 6 inches, respectively, and, calculating the internal diameters of the hollow pillars of equal area, having 6, 10, and 12 inches diameters, we have

Diameters.			Comparative Strengths.		
Solid Pillars.	Hollow Pillars of equal area.		Solid Pillar.	Hollow Pillar.	4.5 × Solid.
	External.	Internal.			
3	6 and	5.196	46.76	209.32	210.44
5	10 „	8.66	279.51	1251.04	1257.80
6	12 „	10.393	529.10	2366.86	2380.95

The numbers in the comparative strengths are simply the 3.5th powers of the diameters of the solid pillars, and difference of the 3.5th powers of the external and internal diameters of the hollow ones. With any particular given length, we have but to multiply the numbers thus obtained by the quotient of the constant 42.35, divided by that length raised to the 1.63 power, and we obtain the comparative number in tons. To apply this truth in aid of the selection of diameters in a proposed hollow pillar, we must calculate the diameter and ultimate strength of a solid column having the given transverse area and length; then, if the given ultimate strength is less than 4.5 times that of the solid pillar, we know at once that the external diameter of the proposed column is less than double that of the same solid pillar.

The last column in the Table shows the excess of 4.5 times the comparative strength of the solid pillar above that calculated for the hollow one of same area, and is about ½ per cent. greater. It will be observed, also, that the thickness of the ring in the several hollow pillars is about one-fifteenth of the external diameter.

It could in like manner be shown that when the external diameter of a hollow column is 3.3 times that of a solid one of same transverse area, then its ultimate strength is nearly ten times greater. Thus, for example, a solid column, 3 inches in diameter, will have the same area as a hollow one of 10 inches external, and 9.53939 internal, and $3^{3.5} = 46.765$, also $(10^{3.5} - 9.53939^{3.5}) = 481.11$, which is only 13.46 greater than ten times the former.

Let the given transverse area of a hollow pillar, at the middle height, be equal to 20 square inches, the structure requiring the column to have a height of 17 ft., and the estimated load, when multiplied by the "coefficient of safety," being 310 tons,—determine by a diagram the external diameter, and, from this last, the internal diameter and thickness of the metal in the intended column.

We may proceed in the following order:—First calculate the diameter of a solid pillar, whose transverse area will be equal to the given 20 sq. inches; and, secondly, the ultimate strength of such a solid column, 17 ft. high, which will give us the first point of the curve. We may then at once ascertain whether the required hollow pillar has an external diameter less or greater than double that of the solid pillar, by observing whether the given ultimate strength required is less or greater than 4.5 times that of the solid pillar; limiting at once our choice of external diameters to those altogether within, or altogether without, this double diameter. Selecting, thirdly, some whole number of inches for an external diameter, we deduce the corresponding value of the internal, and, calculating the strength of this pillar, plot it up, by scale, as an ordinate at the proper diameter on the horizontal line. About three such calculations will be sufficient, and give three points on the curve very near that one which we are seeking, and through them a portion of the curve may be drawn sufficient for the object in view.

First, then, x being the diameter of the solid pillar, we have $0.7854x^2 = 20$ sq. inches.

Hence $x = \pm\sqrt{20 \div 0.7854} = \pm\sqrt{25.4647} = 5.046$ inches.

Secondly, for the ultimate strength we have

$$\begin{array}{rr} \text{To } 3.5 \times \log 5.046 = & 2.4603152 \\ \text{Add log opposite } 17^{1.63} \text{ in Table of lengths} = & \bar{1}.6212224 \\ \hline \text{Product, } 120.65, & 2.0815376 \end{array}$$

If now we double this diameter, which gives us 10.092 inches, we have the ultimate strength of a hollow pillar of that external diameter, and having 20 square inches in transverse section, the height being 17 ft., equal to 543 tons very nearly. Therefore all our trials for dimensions which will give 310 tons must be within 10 inches as external diameter.

It will greatly facilitate our determination of the three points in the curve, if we select some whole numbers for external diameter; we will then take 8, 7, and 6 inches. The internal diameters corresponding to these are 6.2077, 4.8513, and 3.246 inches; and therefore the thickness of metal in each will be 0.8962, 1.0743, and 1.377 inches, respectively.

For the ultimate strength we shall generally have to calculate the 3.5th powers of the internal diameters, as the decimal part will seldom exactly agree with those given in the Table of 3.5th powers; but if the internal diameters have been calculated by logarithms, the labour will be but small, the several operations contributing to the result. Thus, for $64 - 25.4647 = 38.5353$, we have

log 38.5353 = 1.5858588
Divided by 2 for log of 6.20768, the square root, = 0.7929294
Add one-half for the decimal part of 3.5, the index 0.3964647

log of $6.20768^{3.5} = 2.7752529$
answering to the number 596.009.

Then, from the Tables of 3.5th powers we have

$8^{3.5} = 1448.15$
$6.20768^{3.5} = 596.009$

Difference = 852.141, the log of which, 2.9305115
added to log opposite $17^{1.63}$ in Table, $\bar{1}.6212224$

Product, 356.233, 2.5517339

being the ultimate strength in tons, and also, at the proper scale, the vertical ordinate at 8 inches on the horizontal line of diameters in Fig. 39.

In like manner,

$\log (7^{3.5} - 4.8513^{3.5}) = 2.8169436$
To which add log opposite $17^{1.63}$ in Table, $\bar{1}.6212224$

Product, 274.262, 2.4381660

And again,

$\log (6^{3.5} - 3.246^{3.5})$, 2.6697542
Add, as above, $\bar{1}.6212224$

Product, 195.4, 2.2909766

In fig. 39 are plotted at 8, 7, and 6, the ordinates 356.24, 274.26, 195.4, and the curve drawn through their upper points; and although these three are sufficient to obtain the result, it has, nevertheless, been drawn from the first point, and also through that corresponding to 10.09 inches diameter, namely, 539 tons. We now take off 310 on the scale of 200 to an inch, and, the zero being on the horizontal line, lay it off on any two verticals, so that one of the upper points may be outside, and one inside the curve. Drawing a line through these, we cut the curve, and reading by the 30 scale the value of the horizontal portion nearest the left-hand ordinate, we have the decimal part of the sought external diameter. Thus, in fig. 39, on the ordinates at 10 and 6, we lay off 310 by the scale of 200 to an inch; through the points thus found we draw the dotted line cutting the curve, and by the 30 scale ascertain that the point is 0.45 beyond the vertical ordinate through 7, and therefore the external diameter we are seeking is 7.45. To test this by actual calculation, we shall require the values both of 7.45^2 and $7.45^{3.5}$.

log 7.45 = 0.8721563
2

2nd power 55.5025, 1.7443126
For 3.5 power add the above, 0.8721563
and the half of do., 0.4360782

$7.45^{3.5} = 1128.62 = 3.0525471$

To obtain the internal diameter and its 3.5th power,

From $7.45^2 = 55.5025$
Subtract $20 \div 07854 = 25.4647$

Square of internal diameter = 30.0378

log 30.0378 = 1.4776682
Divided by 2 gives 0.7388341, answering to the root 5.4807
Half this last, 0.3694170

$5.4807^{3.5}$ = log 2.5859193 = 385.41,
Again,
log (1128.62 − 385.41 =) 743.21 = 2.8711115
Add log opposite $17^{1.63}$, $\bar{1}.6212224$

Product, 310.7 = log 2.4923339

showing a very close coincidence. The thickness of the metal in this column will be $\overline{7.45 - 5.48} \div 2 = 0.985$ inch, which in practice would be made one inch, and the ratio of the length to the diameter is 27.38 to 1. With a taper or diminution of the diameter from the base upwards of an eighth of an inch on the diameter for every foot in height, we have the top 6.4 inches, and the bottom 8.5 inches, in external diameter.

162. We may now proceed to the second case mentioned in § 153, page 135, namely, to calculate the ultimate load that pillars of given dimensions will sustain. The calculation is simple, but always requires the use of logarithms. If the given diameters and lengths are found in the Table of powers of numbers given at the end of these "Practical Applications," very little remains to be done; and if not, the diagram, fig. 35, will solve the question approximately, or we may use the method of interpolation explained in pages 138, 139.

Let it be required to calculate the load that a cast-iron pillar, 14 ft. long, and having an external diameter of 6.2 inches, and internal of 4.1 inches, will sustain. We have, on performing the required involutions, $14^{1.63} = 73.82$, also $6.2^{3.5} = 593.43$, and $4.1^{3.5} = 139.55$. So that the ultimate strength of such a column will be

$$42.35 \times \frac{593.43 - 139.55}{73.82} = 260.45 \text{ tons.}$$

Mr. E. Hodgkinson, using the indices 3.6 and 1.7, as deduced from his earlier experiments, and the constant 44.16 (pages 87 and 97), makes it 275 tons.

It may be well here to refer the student to the remarks in § 34, page 17. The coefficient of stability in the case of pillars must be even higher, perhaps, than that in other applications of material, inasmuch as a number of pillars are generally combined, at certain intervals of distance, to support a continuous mass, it may be a vertical wall, or a warehouse flooring, &c.; and the slightest irregularity in the foundation below, or settlement above, even though not rendered visible externally, may throw an undue proportion of load upon some one of the series of supports. The ultimate strength of pillars is generally made from six to ten times greater than that of mere equilibrium.

163. In large towns it is frequently necessary to adapt buildings which have been dwelling-houses to purposes of business, in which operation they generally endeavour to leave the upper stories and walls standing, and, having temporarily shored them up, the permanent horizontal beams, either of timber or cast or wrought iron, are introduced, and these are supported at intervals by vertical pillars of cast iron. An instructive account of this operation on a large scale is given by

Mr. Anderson, in the Transactions of the Institution of Civil Engineers of Ireland, vol. v., page 183.

The pillars supporting the back wall of the house, the lower part of which was cleared away to give open space for the cash-office of a bank, as described in the paper referred to, were 26 ft. high, and the external diameter at the base 14 inches, that at the top being 12 inches, fig. 40, 40*a*, &c., the thickness of the ring of metal being 1¼ inches.

Now, this hollow truncated cone will contain the same quantity of metal, namely, 14396.4 cubic inches, as a hollow cylinder 13 inches external diameter (i. e. the arithmetic mean of 14 and 12), and 1¼ inches thick; and the strength of the truncated column is not very much less—about three per cent.—than that of a hollow cylinder of the same weight and external diameter, as appears from Table § 148, page 131, Experiments 3 and 4, and the remarks at the end of page 129. So that we may calculate the strength of this uniformly tapering pillar as if it had been a uniform cylinder, 13 inches external, and 10½ internal diameter. Therefore,

$$42.35\,\frac{13^{3.5} - 10.5^{3.5}}{26^{1.63}} = \text{Breaking weight in tons,}$$

that is
$$42.35\,\frac{7921.4 - 3751.13}{202.49} = 872.4 \text{ tons,}$$

which is more than six and a half times greater than its computed probable load.

In the paper referred to in this example, the author gives an instructive account of the method adopted to keep the core in the centre of the casting, which, with what has been given in pages 109, 111, will point out to the student the practice in this part of the founder's art.

Cast-iron columns are, in practical construction, generally made tapering; and this last example may be taken as a good precedent: in it we have 1 inch of slope inwards upon 312 inches, or 26 ft., of vertical height, say one in three hundred.

Aware of the importance of a flat and firm bearing, the ends of these pillars were turned true in the lathe, and the base, fig. 40*a*, which was a separate casting, 2 feet square, and 3 inches thick, was turned on the upper and planed on the lower face. The foundation stone having been set perfectly level, and dressed to fit the plate exactly, the latter was laid truly in its place, and secured by means of iron dowels run with lead. The cap, fig. 40*b*, was also turned on the under side, and both it and the base had projections ¼ inch high entering the column, to prevent any lateral displacement. The space between the beam and the top of the cap was filled with bars of flat iron, and cast-iron cement tightly driven in.

164. Tredgold, in his "Practical Essay on the Strength of Cast Iron," gives the following examples:—

Let it be required to support the floor of a warehouse by iron pillars, where the greatest load on any pillar will be 70 tons, the height of the pillars being 14 ft. Let x be the required diameter of the pillar; then,

$$42.35 \times \frac{x^{3.5}}{14^{1.63}} = 70 \times 6 = 420.$$

Supposing the coefficient of stability to be 6.
We have, therefore,

$$x = \sqrt[3.5]{\frac{73.8225 \times 420}{42.35}} = \sqrt[3.5]{732.124} = 6.5834 \text{ inches.}$$

Tredgold gives 8½ inches, but does not state the coefficient of stability he applied, and makes his calculation on assumptions which had no experimental basis.

No person practically acquainted with the foundry would now order a solid pillar of either 6.6 or 8.5 inches diameter—half the material in a hollow pillar about three-quarters of an inch thick would give greater strength.

165. The same author also gives the following example: Calculate the diameter for story posts (columns) of cast iron to support the front of a house, the lower part of which is to be occupied with shops. In such a case, each foot in length of frontage may be estimated at 25 cwt. for each floor, and 12 cwt. for the roof, per foot run of front: hence, in a house with three stories over the shops, the extreme load will be, in cwts., $\overline{3 \times 25} + 12 = 87$ cwt. on each foot of frontage. Now, if the story posts or pillars, which are 12 ft. high, be 7 ft. apart, we have the load upon each in tons $7 \times 87 = 609$ cwt. = 30.45 tons; this, with a coefficient of stability of 8, will be 243.60 tons; and, as before, we have

$$x = \sqrt[3.5]{\frac{57.42 \times 243.6}{42.35}} = \sqrt[3.5]{314.} = 5.24 \text{ inches.}$$

If in the former of the above examples from Tredgold, in which the solid pillar would require a diameter of 6.583 inches, we seek the dimensions of a hollow pillar having the same quantity of material, and a thickness of 1.25 inch in the ring, and compare its strength with that of the solid pillar, we have the following result, remarking first, that the rectangle under the sum and difference of the external and internal diameters of a hollow pillar is equal to the square of the diameter of a solid pillar of equal content, each being multiplied by 0.7854 for circular areas, and that the difference of the diameters is equal to twice the assigned thickness; also that twice the lesser diameter, plus the difference, is equal to the sum of the diameters. We have, therefore, calling the lesser of the two z, the sum of the diameters equal to 2.50 + $2z$, and the area of the solid pillar 6.583 inches diameter being

$$6.583^2 \times 0.7854 = 34.036 \text{ sq. inches,}$$

we have

$$0.7854 \times 2.50 \times (2.50 + 2z) = 34.036;$$

dividing and reducing, we have $z = 7.417$ inches for the internal, and consequently 9.917 inches for the external diameter. The ultimate strength of this pillar will therefore be

$$42.35 \times \frac{9.917^{3.5} - 7.417^{3.5}}{14^{1.63}} = 42.35 \frac{3071.5 - 1111.3}{73.82}$$
$$= 1124.56 \text{ tons,}$$

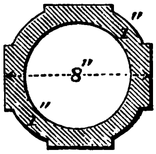
or nearly $2\frac{3}{4}$ times stronger than the solid pillar having the same quantity of metal. Even had we supposed the quantity of metal in the hollow pillar to be but half that in the solid—that is, having only 17.018 sq. inches area—the thickness being as before, 1.25 inch, we would have had its ultimate strength nearly three times the load of 70 tons. In this case,

$$17.018 \text{ square inches} = 0.7854 \times 2.5 \times \text{sum of diameters,}$$

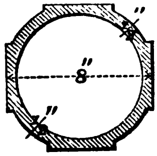
dividing we have the sum of the diameters equal 8.667 inches; hence the external is equal to 5.583 inches, and the internal to 3.083 inches, and the strength

$$42.35 \times \frac{411.184 - 51.453}{73.82} = 206.4 \text{ tons.}$$

166. The woodcuts inserted below represent transverse sections of the cast-iron pillars used in the first Exhibition Building in London, 1851. The elevation and enlarged details of a pillar are given in fig. 41, 41*a*, &c. These cylindrical columns were cast with four flat external surfaces, of a breadth of $3\frac{1}{4}$ inches; the outer diameter was in every column throughout the



building precisely the same, namely, 8 inches; but the interior was varied, so as to adapt the strength of the pillar to the different pressures which they had to sustain in the different parts of the building.



If the external diameter of the pillars had been the dimension altered, variations would have been required in the length of the girders which the pillars supported, and of many other parts of the structure; and the difficulties of the construction, and the time required to design and erect it, would have been much increased.

Each pillar had a base piece or shoe, fig. 41*a*, which spread out into a rectangular plate, either 2 ft. by 1 ft., or 3 ft. by 2 ft., according to the pressure; and this again rested on the top of a mass of concrete on which was spread a thin layer of fine mortar, extending down to the gravel foundation, and having somewhat greater upper area than the bottom of the iron plate. The pillar was bolted down to this bed plate by four screw bolts, four projections or lugs being cast on the bottom of the pillar, and a corresponding number on the base piece. All the columns were turned or faced at the ends, and the surfaces to which they are connected are also faced. The importance of this can easily be understood from the results of the experiments already given, which have shown the advantage in point of ultimate strength of a firm flat bearing for vertical pillars. In another point of view it proved to be no unnecessary or expensive refinement, as it was of the utmost importance in facilitating the rapid erection of the building, and effected a real economy. In fact, it is scarcely saying too much to affirm that the building could not otherwise have been erected in the short space of time granted for that purpose.

Some of the pillars were made use of to convey the rain water down the interior hollow, and in this case the base piece had a lateral pipe, fig. 41*a*, which carried it into the general drainage prepared underneath the building. In such pillars, a piece of canvas covered with white lead was placed between the ends of the column and the surfaces of contact above and below, thus forming a water-tight joint: no packing or adjustment was found to be required to bring the columns perpendicular and in range with one another.

The top of the pillar was provided with projections similar to those on the bottom, already described, and immediately over them. What was called the connecting piece, fig. 41*b*, was fixed on the top of the column, and bolted down upon it by four one-inch screw bolts passing through lugs, the same as on the pillar: this piece was 3 ft. $4\frac{1}{2}$ inches high, and to it were attached the horizontal girders, which were 3 ft. deep, and performed the double office of supporting the galleries or roof in the upper part, and also of giving lateral support to the pillars. It was with the view of giving this support and general stiffness to the building that the depth of the girders was made so much greater than what it would have been with the usual proportion of depth to span, which is from one-twelfth to one-sixteenth, and these were 24 ft. span and 3 ft. deep, or one-sixth of the span. The length of the pillars between the base plate and the connecting piece on the top was 18 ft. 8 inches; and, for purposes of calculating the ultimate strength, this is the length we must take, as this portion was in the position of a vertical pillar firmly fixed at each end. In the two-story part of the building, a column 16 ft. $7\frac{1}{2}$ inches was placed, and bolted at the top of the connecting piece before mentioned; and this shorter pillar itself carried a connecting piece of the same depth as that on which it rested. In the three-story parts of the structure, another column, 16 ft. $7\frac{1}{2}$ inches high, was placed on the second connecting piece, and, as before, itself carried a third connecting piece; to each of these last, as to the lowermost, horizontal girders were firmly attached, supporting either a gallery or the roof, and acting as giving lateral stiffness to the whole building, and holding the pillars at their ends securely, and in a vertical position. The respective heights from the ground-floor to the top of the each range of girders were 22 ft. 2 inches, 42 ft. 2 inches, and 62 ft. 2 inches.

167. If we neglect the area of the metal contained in the four fillets, which may be taken as equivalent to $4 \times 3\frac{3}{8}$ inches $\times \frac{5}{32}$ inch, and therefore amounts to 2.1 sq. inches, we have in the case of every pillar an external diameter of 8 inches, and an internal varying from $5\frac{3}{4}$ inches, which corresponds to $1\frac{1}{8}$ thickness in the ring, and was that of the lowest and strongest pillars, down to 7 inches internal, answering to $\frac{1}{2}$ inch thickness, as in the second woodcut. In the strongest pillar we have the ultimate strength as follows, calling the length 19 ft., which is on the safe side:—

$$42.35 \times \frac{8^{3.5} - 5.75^{3.5}}{19^{1.63}} = 42.35 \times \frac{1448.15 - 455.87}{121.44} = 346 \text{ tons};$$

and upon these strongest columns, situated in the nave, the actual weight would be only 60 tons.

If we calculate the strength of the pillars 1 inch thick, as shown in the first woodcut page 166, which have therefore 6 inches internal diameter, we have the ultimate strength,

$$42.35 \times \frac{1448.15 - 529.08}{121.44} = 320.5 \text{ tons.}$$

Those ½ inch thick, page 167, which are therefore 7 inches internal diameter, have an ultimate strength of

$$42.35 \; \frac{1448.15 - 907.49}{121.44} = 188.5 \text{ tons.}$$

If we suppose that the area of the four flat faces or fillets external to the circle, having 8 inches diameter, was placed in a uniform ring around and concentric to it, we may calculate the addition it would, when thus placed, make to the external diameter of the pillars, and thence compute the additional strength it gave. Let x be this increased diameter; then $0.7854\ (x^2 - 8^2) = 2.1$ square inches, so that

$$x = \sqrt[2]{\frac{2.1}{0.7854} + 8^2} = \sqrt{66.673} = 8.165 \text{ inches.}$$

and $8.165^{3.5} = 1554.4$, which will have to be used instead of $8^{3.5} = 1448.15$ in the several calculations of the strength; thus the column which was 1⅛ thick would have a breaking weight equal to

$$42.35 \times \frac{1554.4 - 455.87}{121.44} = 383 \text{ tons.}$$

The probability, however, is that the strength of the column would be best represented by supposing it to be a uniform cylinder of 8 inches diameter on the outside; at least, Mr. Hodgkinson's experiments on the varied forms of pillars, § 145, do not give any presumption in favour of the ultimate strength of those different from the uniform hollow cylinder.

168. The calculations in the preceding sections will suffice to point out the method of determining the requisite thickness of the metal in the shafts of cast iron columns under given circumstances. It may now therefore prove useful to study, by means of a selection of approved examples, the various methods by which engineers have combined the shaft and the other essential parts of the column, such as the base piece, the capital, the connecting piece, &c.; so as to give a design, with all its details, suited to the particular objects of the structure. In some of the previous examples, indeed, this part of the sub-

ject has been unavoidably touched upon, when treating of the ultimate strength of the shaft.

169. Commencing then—in the same order as we build—with the base piece and its special form and fastenings, we may observe generally, as to this part of the column, that there are two extreme cases of pillar-supported structures:—One in which, as in a covered landing stage for passengers at a pier, a good shed at a quay side, or a shed at the junction of different lines of railway, &c., the pillars support a roof without any surrounding side walls; the other case occurring when, as in a warehouse or manufactory, solid walls enclose the structure on every side. In the former structures the pillar has but little insistent vertical weight to support, and may in truth, when calculating its dimensions, be regarded as a vertical beam, fixed at one end, that is, at its base, and loaded at the other, namely, where the roof is attached to the upper part of the pillar, the horizontal force of the wind acting on the roof so as to produce a transverse load upon the several columns, the base piece consequently requiring to be fixed to the foundations in the most secure manner; otherwise, under the influence of violent storms of wind, the whole structure would be liable to be thrown down or carried away laterally.

In the second case, that is, when the building is closed in on all sides by substantial walls, the internal lines of columns are performing their proper duty of supporting great vertical pressures; but in designing the details of the base, it is only necessary—after providing sufficient bearing area—that it be truly bedded on its foundations; and if any dowel-pins or screw-bolts are made use of, it can only be for the purpose of obviating the danger of some lateral displacement during the process of erection.

170. The building, constructed for the International Exhibition of London, in 1851, was a structure in which cast iron pillars formed an essential feature, and it affords an instructive example of the method of applying them to answer different purposes by varied adjuncts and details. We place it first among the following examples as having, in the sections immediately preceding § 166, &c., p. 166, given the calculations of the ultimate strength of the form of the pillars employed in it.

The careful study of all the details of any work such as this now under our notice cannot fail to be useful to the beginner. The most inventive mind is oftentimes more dependent on precedent, when designing, than it may be willing to acknowledge, or even be aware of, and is most certain of fulfilling its own particular object when fully acquainted with all that has been effected by others in circumstances more or less similar,—this course being, in fact, to make the experience of others our own when working out any required design.

171. The Commissioners appointed, in 1850, for carrying out the idea of an International Exhibition, had, in answer to their advertisement, received about 240 plans for the required building, from nearly every civilized country as well as from Great Britain; but, rejecting these *in toto*, they proceeded themselves to frame a design. Valuable time was thus lost; and, as their own design would have involved, amongst other things, the laying of about 17,000,000 of bricks, chiefly during the next succeeding winter, they were undoubtedly in a position of some difficulty, if the long fixed day of opening, namely the 1st of May, 1851, was to be adhered to.

The examination of competitive designs by those who have invited them has often resulted, both before and since, in a similar course of proceeding, and always with the same resulting well-founded dissatisfaction; for it does seem almost impossible for a body of men, fresh from the close study of a number of different plans, to sit down and design one more, with the same general objects in view, without adopting into it most of those points which may have struck them as best in the several different drawings that have been intrusted to them. But at this stage the Royal Commissioners were saved both from their angry critics, and from their own plans being carried out, by the sudden appearance of Sir Joseph Paxton's design, and its immediate adoption.

The design that had been elaborated by the Commissioners themselves is given in the "Civil Engineer and Architects' Journal" for 1850, p. 209, with a map of the selected site.

In seeking for tenders for the execution of their own plan, it was stated that they would be glad to receive any suggestions from intending contractors. In their own words, proposals for "methods of construction other than those shown upon the drawings would be entertained," if made in a *bonâ fide* form, which having through Messrs. Fox, Henderson, and Co., been shown to Mr. (now Sir Joseph) Paxton, he very shortly produced the design, founded on the model of that which he had already erected at the great Conservatory at Chatsworth; which was at once accepted, and, with some alterations, finally carried out.

172. This Exhibition building may be truly said to have

inaugurated a system of iron architecture, and has no doubt had considerable influence in developing that extended application of iron which marks the progress of the constructive arts at the present day.

The materials used in this design—to the exclusion of brick and stone—were iron, wood, and glass; and it is remarkable that no large pieces of either material were employed. The heaviest pieces of cast iron were the girders, none of which weighed more than a ton. The building may be described, generally, as consisting of a framework of columns, united by girders or trusses, and covered with glass.

The form and character of the ground having to a great extent dictated the outline of the proposed plan, we may therefore state that the site consisted of a portion of Hyde Park, in form nearly approaching a parallelogram, of which the sides are about as four to one, being 2300 ft. long by 500 ft. broad, measured on the central axes; fronting north and south, and having its sides east and west, and with an almost uniform fall of one in 280 from west to east. As the building was only intended to be temporary, it was determined to preserve the trees, a lofty line of which stretched about half across the centre of its length; and two clumps of smaller trees stood near the northern boundary, on the east and west of the central groups. It will be found that the design was well adapted even to this extraordinary requirement,—the lofty transept arching over the higher trees, and the open courts preserving the younger clumps on the northern side.

173. The general plan of the ground floor is shown in fig. 48, Plate VII., at a scale of 120 ft. to an inch. Its form will be seen at once to be that of a rectangle, in length over all 1850 ft., and in breadth about 408 ft., on the north side of which an addition was formed, 936 ft. long, and 48 ft. wide, to give a further space for the machinery in motion, which dimensions were found to be most suitable to the boundary of the piece of ground described above, which was, though after great opposition in Parliament, obtained for it in Hyde Park. It will be perceived that a main central avenue, or nave, extends the whole length of the building, in width 72 ft.; and by a reference to fig. 49, which is a transverse section, looking west, through the vertical plane on the line A, B, in fig. 48, its height is shown to be 64 ft. This nave is intersected nearly at the middle of its length, and at right angles, by a transept, also 72 ft. wide, but covered with a semicircular arched roof of ribs of timber springing at the level of the

horizontal girders spanning the nave, and giving this part of the structure an extreme height of 104 ft.

On each side of the nave a double row of columns, 24 ft. apart, forming long narrow aisles, extends the whole length of the building, except at the crossing of the transept; and outside these, on both sides, are the 48 ft. avenues, their height being 40 ft.; again we have a double row of columns, 24 ft. apart; then another 48 ft. avenue, 22 ft. high; and, lastly, a third double line of columns, 24 ft. apart, the outer row of which forms the chief support of the external walls—if the word may be so applied—of the building on the ground floor. Whatever different distance apart these pillars are placed in the transverse lines of the building, fig. 49, whether 72 ft., or 48 ft., or 24 ft., they are all 24 ft. apart in the direction of the length, as is shown in the portion of the side elevation, fig. 50, except only where the transept crosses; and even at that part the two outside lines on each exterior face are absolutely continuous for the whole 1850 ft. The girders in this direction are all, therefore, 24 ft. span, and are formed of cast iron, with the exception of the outside lines in the one-story part of the building, which are of wood.

The analogy of these general arrangements to the prevailing form of the cathedral structures of the middle ages was so strong, that the names of nave, transept, and aisle were universally and at once applied to the analogous parts of the exhibition building. A light iron railing enclosed the building on all sides, at a distance of 8 ft. from its exterior, and outside that a broad gravel walk was placed.

174. The six pairs of dotted diagonal lines in fig. 48, across the width of the main central nave, and lettered E, E, &c., show the position of the horizontal bracing attached to and placed immediately under the 72 ft. wrought iron girders, which span that opening at a level of about 61 ft. from the floor. These bracing rods of wrought iron were the diagonals of squares of 72 ft. in the side. Other horizontal bracing was fixed in the squares of 24 ft. in the side, and is shown by dotted lines at F, F, &c., being introduced at each end of the nave and transept. The places at which vertical diagonal bracing was introduced are shown in fig. 48 by letters *a*, *a*, &c., and in fig. 49 by dotted diagonal lines, lettered also *a*, *a*, &c. This bracing was in vertical planes, both parallel and at right angles to the central axis of the building. Those only at right angles to the centre line are capable of being shown by the

dotted diagonal lines in the transverse section, fig. 49; those across the 48 ft. avenues are placed at the transverse galleries, or at the several pairs of stairs.

It will be perceived that the main horizontal diagonals, E, E, and also the vertical bracing at the transverse galleries and stairs, were introduced on the same transverse lines of the building, thus constituting these parts as buttresses, securely stiffened, aiding in upholding the remaining portions of the structure against any deformation to which it might be liable from the prevailing rectangular form.

175. In fig. 49 is given a cross section on the line A, B, fig. 48, at a scale of 30 ft. to an inch. It exhibits one of the great transverse frames of the building, which are placed parallel to each other, and at successive distances of 24 ft., measured on the longitudinal axis. These frames consist of 12 vertical lines of pillars, and 17 horizontal girders, arranged symmetrically on each side of the central line. The pillars are placed in pairs, 24 ft. apart, the pair immediately on each side of the central line being at a clear distance of 72 ft., which is the width of the nave, and all the others at distances of 48 ft., so that it has been likened to the arrangement of a timber bridge consisting of a stack of piles, acting as a pier, on which the horizontal beams are laid.

Each of the four central pillars,—the total number of the same height in the nave being 316,—consists of seven separate castings, bolted together, as shown also in Plate VI., fig. 41, namely, a base piece resting on a mass of concrete, a pillar shaft of the ground tier, a connecting piece on the level of the girders under the galleries, a pillar shaft of the second tier, a second connecting piece, a pillar shaft of the third tier, and, lastly, a third connecting piece on the level of the roof trusses. The next two pair of pillars on each side of the central axis consist of five separate castings; and the ten outside pairs of only three, namely, the first five, and first three enumerated above, when speaking of the central and highest pillars.

The height of the base pieces varies with the transverse slope of the ground surface; the height of the lower tier of shafts of pillars is 18 ft. 8 inches; the connecting pieces in every part are the same height, namely, 3 ft. 3½ inches; and the second and third tiers of pillars are each 16 ft. 7½ inches high.

In fig. 49, we also see at once the relative vertical height of the several tiers of the structure, and their decreased breadth in each successive story; the nave—again resembling in this the cathedral type—being the highest. The arrangement in pairs of columns, 24 ft. apart, is even more distinct in the transverse section than in the plan, fig. 48.

The first columns on either side of the central line have evidently to support a greater load than any of the others in the same transverse plane,—this load, beginning at the roof downwards, being made up of the half of the 72 ft. girder and roof on it, half the 24 ft. girder, and half of its roof, next the half 24 ft. girder on the level of the roof of the second tier or story, and, finally, half the gallery, with the goods exhibited, the visitors, and the supporting girders. All these transverse widths being multiplied by 24 ft., that is, for 12 ft. on each side of the transverse plane, in the longitudinal direction, give the area to be supported by each column, and such would be the load in a perfect structure; but unavoidable irregularities may throw a much larger proportion on some one column, relieving others to the same extent. What the additional load thus produced may be, no possible rule can decide; at a maximum, it may be considered as about double, and must be provided for by the column having a strength five or six times greater than its utmost computed load.

The load on the successive columns, proceeding from the centre on each side, may be calculated in the same manner. All parts of the column were duly proportioned to these loads; the thickness of the metal in the shafts and connecting pieces increased in each successive tier as we descend; and, on the level of the same story, the strength or thickness of metal increased in each pair from the outside pair to those bounding the nave, on either side.

The area of the base plates, and of the supporting concrete, increased also from the outside, commencing at an area of 2 square feet, or 2 feet by one, up to 6 square feet, or 3 ft. by 2. The heaviest loaded pillars were evidently those which supported the semicircular wagon-headed roof of the transept; and the severest load on these was evidently at the angles of intersection of the nave and transept, lettered H, H, &c.; and at this point the thickness of metal in the column was 1¼ inch, and a duplicate arrangement of twin columns was adopted, as shown by the two dots on the plan, fig. 48; special castings united these at the level of the several connecting pieces. The load upon the pillars on the level of the ground, at this point of intersection, was constituted not only of the half galleries and 24 ft. columns, together with the visitors, as a moving load; but also of half the roof of the transept, and one-fourth of the weight of that portion of this semicircular roof which covered the square of 72 feet formed by the intersection of these two main avenues. A pair of the strongest wrought iron girders being placed across the nave at this point on each side of the transept, which between them carried the weight of the semicircular roof for 72 ft. of its length; by this means there was an unbroken length of arch for the whole 408 ft. of the transept.

The dotted lines, *m*, *m*, on the general plan, fig. 48, show the rectangular spaces, which throw their weight, respectively, upon the double columns at the angle and upon the contiguous pillars, the former being a square of 48 ft., and the latter rectangles 24 ft. by 48.

The flooring, it will be seen, fig. 49, was carried on the level transversely; and the ground surface having a fall to the south, advantage was taken of this to give the building on that side the appearance of a raised foundation, which was faced with green sod. The external beauty of the principal façade was thus greatly increased.

176. The arrangement of all the principal dimensions in multiples and submultiples of one primary length namely, 24 ft., is deserving of remark. It not only facilitated and economized all the operations of construction by the frequent repetition of the same parts, but also added to the beauty and symmetry of the structure when finished, long vistas being seen between the columns in the diagonal directions, as well as in the longitudinal and transverse views. The mistakes, and consequent delay, attending the adoption of irregular dimensions, were thus avoided; the lines of the building were rendered agreeable; and nearly uniform areas of roofing could be conveniently arranged for drainage through the hollow columns; and, to use a workman's phrase, "the work would run off all the better for it." The width of the transept and main aisle was 72 ft., that is, 3 × 24; that of the side aisles 48 ft., = 2 × 24. The total length amounted to 1848 ft., that is, 77 × 24; and the breadth, 408 ft., which is 17 × 24; all these distances being from centre to centre of columns; and so of many other dimensions, some being multiples, and others submultiples of the number 24. The choice of this number originated from its being found that sheet glass suitable for the roof could be procured 10 inches wide, and 49 inches long, weighing 16 oz. per square foot (which gives a thickness of about $\frac{1}{10}$th of an inch), but that any greater length than this would be much more expensive, from the increased difficulty of manufacture. Glass of this

length placed on a slope of 2½ horizontal to 1 vertical was found suitable for a ridge and valley roof in which the pitch, or distance from centre to centre of gutters, is 8 ft. Three times this distance, or 24 ft., as the interval of the centres of the columns, was considered to give a convenient length for the cast iron girders, namely, 23 ft. 3¾ inches,—that is, the external diameter of the columns being 8 inches, and ⅛th of an inch being allowed over at each end of the girder. Some such practical consideration in the details is often found to regulate eventually the dimensions of even the largest works.

When this structure we are describing was re-erected at Sydenham, the same unit was adopted in its dimensions, the length being 1668 ft., the breadth 384 ft. with wings on each side, 576 ft. long. Three transepts intersect the length, the central being 120 ft. wide, and the other two 72 ft.; all which numbers are multiples of 24.

177. It is difficult to imagine any other design which could have fulfilled all the conditions required in this building. Its vastness may be inferred from the simple statement that on completion it covered a roofed-in ground area of 18 acres, and contained a volume of about 33,000,000 cubic ft., while the total surface of the ground floor and galleries, added together, amounted to very nearly 1,000,000 square ft. It was, moreover, imperative that the building should be erected within a period of ten months, the Commissioners having made engagements to open the Exhibition on the 1st of May, 1851, and the contractors not being in possession of the ground until the 30th of July, 1850, could not therefore commence the works previous to that date. The unusual stipulation—which must influence the design to be adopted—was also added to the contract, that the whole structure should be removed within seven months after the close of the Exhibition.

The total ultimate cost was £200,000, being about £25 per 100 square feet of ground floor, equivalent to 5*s.* per square foot, or 1½*d.* per cubic foot of the whole volume of the building. About 4000 tons of cast iron were used, and 400 tons of glass, together with 1200 loads, *i. e.* tons of timber. The number of pillars was 3300; of cast iron girders, 2150; and of wrought iron roof trusses or girders, 372. The actual period in which the works were completed was about 190 working days. The greatest number of men employed at the building at one time was 2000, and, including those in establishments supplying materials, would probably reach 5000. Returning now to our proper object, that is, the details of the pillars:

177. The base-piece, already mentioned, § 166, is shown in fig. 41, *a*, at a scale of 2 inches to a foot; and in fig. 41, at ¼ in. to a foot.

It consists of a short pillar, B, identical in section with that of the shaft of the column which it supports above, and of varied height according to the form of the ground underneath the floor line. It spreads out at the bottom into a flat plate, which increases in area from 2 ft. long by 1 ft. wide on the outside longitudinal row, up to 3 ft. long, and 2 ft. wide, on the inner rows, according to the different weights they had to sustain.

The connexion between the vertical part and the horizontal plate is strengthened by gussets, or shoulders, shown at C, only two, in the direction of the length, being used in the smaller sizes, two additional ones being applied across the breadth in those of larger area. In the base pieces placed under such of the columns as were designed to carry down the rain water from the roof, that is, the inner row of each tier, marked R, in fig. 49, horizontal outlet pipes, D, D, were cast, 7 inches outside diameter, having ½ inch thickness at the column, and ⅜ths at the end: these conveyed the rain into drain pipes, and ultimately into brick culverts, leading to the Kensington sewer. The outlet pipe was supported underneath by a vertical rib between it and the lower plate, shown at E, fig. 41*a*, and in plan by dotted lines.

It will be perceived from the drawings that the thickness of the horizontal plate, the duty of which is to disperse the pressure of the pillar over the surface of the concrete, diminishes towards the edge on each side by planes sloping downwards from the centre. The diagonal lines on the plan represent the horizontal projections of the intersections of these inclined surfaces on the top of the plate. This arrangement shows a judicious distribution of the metal. If the plate had been a uniform thin sheet containing the same amount of cast iron, it is more than probable that it would have bent downwards immediately under the pillar where the insistent weight is applied. Setting aside the danger of fracture to the plate from this deflection, it is evident that the pressures on the surface of the concrete would not be uniform, and thus one great object of a base plate be defeated, neither would the pressures be vertical in direction, except immediately under the pillar; but, being perpendicular to the curved surface of the plate, would have a tendency to split the mass of concrete, and force it outwards.

178. The bottom plate of the base piece, described above, is set upon a mass of concrete projecting about six inches on all sides, and having on its upper bed a thin layer of fine mortar. This concrete rested below on a stratum of gravel which extended under the whole building, the depth to which it was carried varying from one to four feet; but in some few places, where the ground was bad, a much greater depth was employed. This material, which is formed of lime and gravel in proportions varying from one of the former to between six and ten of the latter, may be considered to have a twofold object, first, that of extending the superficial area of the base, and so diminishing the unit pressure on the ground arising from the weight of the structure it supports; and, secondly, of supplying an economical means by which to reach a sufficiently firm foundation on the natural ground, from the upper level of the intended base; being also laid in its place in a loose disintegrated state, it takes, when set hard, the exact form of the surface of the ground, and becomes, in fact, a monolithic mass, well suited for its requirements in every respect.

The area of the lower bed of the concrete was determined, as that of the base plate, by the estimated weight of the superstructure at that part; and it was so proportioned that, allowing for every possible contingency, under no circumstances should a pressure greater than 2½ tons per foot, superficial, be brought to bear upon the gravel. The length of the plate was always placed in the direction of the width of the building, that is, in its least dimension, with a view to obtain greater lateral stability in the structure.

It will be observed that no screw-bolts for holding down the base plates were employed; but in this, which must have greatly expedited the rate of progress in the construction, a sound judgment was used; the transverse section of the building, fig. 49, pointing out the great comparative base of the whole structure in relation to its extreme height, which was about six to one, will serve to demonstrate its perfect security from all danger of being overturned or uprooted, as a whole, by any lateral force. On the other hand, this building cannot be strictly included in the second case of pillar-supported structures (*vide* § 169); for the outer walls were formed of lines of columns, to which alone were attached the boarding and windows enclosing it on all sides; and therefore the pressure of the wind was chiefly resisted by the transverse strength of the outer and ground tiers of pillars, and by the stability of the base pieces.

179. The upper part of the base piece was constructed with a horizontal flange, and four lugs having holes, through which were passed the screw bolts whereby the pillar was firmly held in position; the form of this flange is shown in fig. 41, *f*, which also gives the precisely similar form of the bottom of the shaft of the pillar; and both being turned or faced in the lathe, no fitting was therefore required in the erection of the pillars, which on being severally placed upon the top of the base plates, the four bolts passed down, and the nuts screwed up, the operation was completed.

The method of uniting these two parts of the columns, just described, was followed in every part of the building, except in the outer line of pillars in the one-storied portions, that is, in the north and south elevations on the ground floor; and here the pillar and base plate were united by a socket joint, shown in fig. 41, *t*, the inside surface of the pillar being turned in the lathe of a slightly conical form, and the upper and outer part of the base piece being also turned to the same angle of cone, so that, when the latter was bedded truly on the concrete, the pillar was simply dropped upon it, and the joint formed in a very rapid and economical manner, compared with the four screw bolts used in other places, and admissible in this row of pillars, as they had to bear so small a load, which was only the half of the roof and roof girders of one square of 24 feet. The external fillet cast upon the foot of the pillar outside the socket was also better adapted to carry on the lines of the timber work, filling in the space between these external columns, than the flange with four lugs and screw bolts used elsewhere on the ground floor—but always concealed by the ornamental light cast iron plinth affixed—would have been.

The common surface of the flanges, in which the foot of the pillar and the top of the base piece bore on each other, was placed 3¾ inches, in the clear, above the floor line; the screw bolts were thus accessible, and the timber work of sleepers, joists, and planking, did not require that troublesome fitting and scribing to the pillars which would have been necessary if it had been placed at that level.

The base plate was the only part of any column from the ground to the roof, which varied in height in different places on the level of the same story. An inspection of the transverse section, fig. 49, will show the reason of this: the ground having a uniform fall from the south to the north, and the line of the floor being horizontal in the transverse section, it follows that the vertical distance between these two lines must be increasing at every line of pillars. As the floor line, in the longitudinal direction, was carried nearly parallel to the ground line, which had a uniform fall of 1 in 280 from west to east, the height of each longitudinal row of base plates was the same, so that no mistake or confusion in the casting or erecting was likely to occur, with ordinary care. The total number of these base pieces was about 1060, corresponding with that of the pillars in the lower story. The average area of the base plates being about 4 square feet, the building therefore rested upon about 4240 square feet of foundation surface.

180. The shaft of the pillar has been fully described in § 166, 167, with woodcuts showing the horizontal sections. The four flat faces of the fillets were placed parallel to the main axes of the building, the lugs for the connecting screw bolts being on the diameters making 45° with the same lines. The pillars are shown in fig. 41 in full elevation at a scale of 4 feet to one inch, in a three-storied part, parallel to the longitudinal vertical plane, being the columns lettered Z, Z, on each side of the transept, where from the letters *a*, *a*, we see that diagonal bracing was used. All the shafts of the ground tier of pillars were of the same height, namely, 18 feet 8 inches. Their external form was suggested by the late Sir Charles Barry, and was well adapted for its purpose, mechanically as well as artistically; for while it presents a pleasing variation from the ordinary circular form, the different flat bands upon it afford surfaces well suited for the connexion and attachment of the necessary girders, &c.

The form of the shaft above described has been adopted by M. Alexis Barrault, in the permanent exhibition building in Paris, named Palais de l'Industrie. The following are the words of high commendation in which, after his description of the column, he speaks of its external elevation:—"Cette forme, que nous avons empruntée au Palais de Cristal de Londres, convient sous tous les rapports aux constructions en métal: elle est élégante, tout en étant facile à exécuter, et se prête très-bien à toute espèce d' assemblages."

It very rarely occurs that any subject of an artistic character originating in England has been imitated in Paris as we find this has been.

181. Diagonal bracing, in vertical planes, is introduced at intervals between the columns, for the purpose of adding to the stiffness of the structure. In fig. 41 is shown a part elevation of the vertical diagonal bracing in three tiers of

columns near the transept, and immediately underneath the plan, or horizontal projection, of the same. It consists of four pair of round iron rods, one inch in diameter, in each rectangular space; these are shown, in elevation at the letters, N, N, &c., in the several drawings, and in plan in fig. 51. Each rod is cranked, and forged with an eye at one end, by which it is attached to the pillar, and is screwed at the other to receive a nut. The bolts D, D, figs. 41, *a*, and fig. 46, which fasten the columns to their connecting pieces, are lengthened so as to pass through these eyes, and thus secure the rods, N, N. The screwed ends of the rods enter a circular cast iron adjustment-plate or hook, M, fig. 41, where they are secured by nuts. Fig. 41 shows the connexion with the adjustment plate drawn to a larger scale. In order to adapt the bolts D, D, uniting the pillars and connecting piece, for the additional duty thus required of them, they are lengthened, and made to pass through four wrought iron clips, 1¾ inch wide, by ½ inch thick, which embrace the pillars on all sides. Fig. 43 shows, at a larger scale, a side view, and fig. 44, a plan of the clips in their proper positions relatively to each other, and a single clip separate from the others is also drawn in fig. 45. The horizontal section of the pillar embraced by the clips is also shown in fig. 44.

The diagonal bracing of the columns on the ground floor is the same as that of the upper ones, with the exception of the attachment at the lower part: in this case the bolts are still further prolonged to support the clips, of which there are eight of 1¾ inch wide by ⅜th inch thick, forming a double ring round the column. The lower ends of the diagonals are thus about 2 feet 6 inches above the floor line. When all these parts have been duly fitted, the requisite strain is brought equally on the several diagonal rods by tightening the nuts within the adjustment plate upon the screwed ends of the rods. The inner surface of the ring, it will be perceived in the drawing, has a flattened portion for the nut to work upon. Finally, two light cast iron ornamental caps are placed over the sides of the adjustment plate, and these are connected together, and also fixed to the plate, by screwing one of the caps on a wrought iron pin, previously screwed into the centre of the other cap. In order to reduce the width of the ring or adjustment plate, the diagonal tie bars were slightly convergent from the clips to the point of intersection at the nuts, as shown by dotted lines in the plan, fig. 44, and in the plan underneath, fig. 41.

The rectangular spaces formed by the pillars and girders were thus divided into several triangles with fixed sides of unalterable length, and the permanence of the figure thereby secured, the triangle being the only geometrical figure whose angles are determined when the sides are given, and whose form is therefore inviolable,—this being the same principle of construction which gives the required stiffness to scaffolding, centring, timber ships, gates, and framing of every kind, whether for temporary or permanent use.

The vertical diagonal bracing, as already mentioned, was introduced at the intersection of the nave and transept, and at six equidistant transverse planes, for a width of two bays of 24 feet in the lateral or 48 feet aisles, and is shown in fig. 48.

The horizontal diagonal bracing, which was also used, was introduced, as already mentioned, under the roof girders in the squares of 24 feet along the ends of the nave and transept, and at six places under the roof of the nave, in squares whose side was 72 feet,—the diagonals intersecting in the same planes in which the vertical bracing was situated. These are shown in the plan, fig. 49, by dotted lines.

182. The connecting piece, which, as its name implies, unites not only the pillars of the several stories to each other in the vertical line, but also supports and joins into a compact framework the horizontal girders in every direction, is represented at different scales; in fig. 41, *b*, with the pillars and girders attached, at one inch to a foot, and in fig. 46, at 2 inches to a foot; in fig. 47 a sectional plan is shown, taken through the line *w*, *z*, on the elevation above, all the parts of the attached girders shown in fig. 46 being omitted in fig. 47. This part consists of a short pillar—identical in horizontal section with those of the columns which it connects—and an upper and lower flange, T, T. In each of these flanges are four bolt holes answering to those in the lugs at each end of the pillars. The ends of the connecting piece being turned or faced in the lathe, in the same manner as those of the pillars, formed with them a perfect joint when laid in place, being fastened together by screw bolts dropped through the holes when brought into the proper vertical line. The dotted lines on the sectional plan, fig. 47, represent the lugs on the top of the supporting pillar beneath.

The flanges, at each end of the connecting piece, also carry four projections, or snugs, G, G, figs. 46 and 47, serving, below, as brackets to support the girders; and by the hooked form of the several ends acting, both above and below, as clip to retain them in place, and obviate any tendency to move in the direction of their length. In order to hold them [illegible] in the transverse direction there is a groove left in [illegible] inch wide, and ¼ inch deep, [illegible] In order to render this groove effec- [illegible] upon each end of the bottom [illegible] tenon-like projection, un- [illegible] into the groove left in the [illegible] end of the upper surface of [illegible] sunken-groove, H, correspond- [illegible] hooked clip. When the girder [illegible] iron key of sufficient depth to [illegible] in firmly; the same operation [illegible] end, it is evident that the vertical [illegible] secured against any transverse force [illegible] movement. In order, however, to tie [illegible] columns in the longitudinal direction also, [illegible] to the plane of the girder—and shown [illegible] fig. 46, and also in fig. 41, *b*, is driven, at [illegible] bottom, in the space between the inner face [illegible] clips and the corresponding face of the projec- [illegible] end of the girder; the same arrangement being [illegible] each end attachment. The keys used for fixing [illegible] which lie transversely in the building are all of [illegible] iron, and similar keys are used with those which lie longitudinally, for distances of 200 feet from each end of the building, and for 200 feet on each side at the intersection of the transept. In the two intermediate portions, each of the length of about 500 feet, keys of oak are employed, so as to allow a small degree of elasticity, and admit of the expansion and contraction of the metal.

183. The connecting piece was modified according as it had to unite two, three, or four girders, and also for external columns, &c., that which has been engraved being intended for the junction of four girders. It will be evident that each connecting piece must be attached to the pillars below it previous to the operation of placing the girders in their respective positions around them, and therefore such a mode of fastening the girders to the connecting pieces must be adopted as will admit of their being passed laterally into position at each end, when lifted to the proper level; the method just described plainly fulfils the several conditions of the case.

The girders of cast and wrought iron which were used in this structure will be introduced as examples of beams in Part II. of this work.

In the one-storied portions of the ground tier these connecting pieces merely support and unite the roof girders; but in all the other places a second pillar shaft, 16 ft. 8 inches high, and having flanges identical as to lugs and bolt holes with the connecting pieces, is placed and bolted down. On the upper flange of each of this second tier of pillars another tier or stratum of connecting pieces is fastened by screw bolts and nuts, which in the two-story portion unite, and sustain the roof girders; and in that of three stories, besides performing the same duty to the stiffening girders (which bear neither gallery nor roof), they have the third vertical tier of pillars bolted down on their upper flanges, exactly as the tier below was on its connecting piece. These pillars were the same length, namely, 16 ft. 8 in., as those immediately below them. Finally, the third stratum of connecting pieces is bolted on the upper flange of the third tier of pillars, and in this position serve to unite and support the roof trusses of the nave and its two side aisles.

184. The whole seven different pieces are shown united in fig. 41. The column on the right-hand side represents the octagonal castings attached, which cover all the flanges and screw bolts in the interior of the building, namely, the plinth on the level of the ground floor; the capital immediately below the bottom of the first girder, over the flanges at the bottom of the connecting piece: a similar capital is placed below the second and third tiers of girders, covering the lower flanges of their connecting pieces; also, above the first and second tiers of girders, the upper flanges of their connecting pieces are covered and ornamented by castings the same as the capitals, but inverted,—the thickness of the metal in all of them being $\frac{3}{16}$ths of an inch.

The plinth, or base, shown in figs. 52, 53, 54, Plate VII., scale $1\frac{1}{2}$ inch to the foot, is in two castings, which join at the angles in such a manner that three sides of the octagon are in one casting, and five sides in the other; they are united by two $\frac{5}{8}$th inch bolts and nuts, *d d*, passing through internal flanges, *e e*, whose position is shown in the plan, fig. 53, and in the elevation of the joint, fig. 54. The flange *e* is set back $\frac{1}{16}$th inch from the plane of the joint, and has a small projection or chipping piece at *b*, which projects $\frac{1}{16}$th inch, so as to lie in the plane of the joint. The flanges will thus come to a firm bearing without danger of breaking when the screw bolt is brought up tight, and the external lines are rendered quite true.

The band, or capital, which embraces the columns, is shown in elevation and plan, figs. 55, 56; it is cast in two equal parts, which are united by two small screws, *d d*, on each side, passing through the castings, and entering a small strap of wrought iron, *e*, placed transversely to the joint in the interior of the casting. Notches are formed at the sides of the castings, where the girders pass through them. In those places, also, where diagonal tie rods are introduced, the castings were specially notched to allow them to pass through.

185. A question of great practical importance may here be asked, namely, would it not have been possible to have constructed the higher columns with a smaller number of pieces than seven? As erected, we have six pair of joint surfaces to turn in the lathe truly perpendicular to the central axis, and four screw bolts to provide for each pair. It was not that any inconvenience or difficulty in casting, or in the carriage and fitting up, would have arisen from the greater weight of fewer pieces; but it will be observed, in explanation of the plan adopted, that the only external alterations of the castings of a column required in the various different parts of the building were in the connecting pieces and base plates—the former, according as they had to unite two, three, or four girders; and the latter, according to the depth of the ground below the floor line, the area of base required, and whether or not intended for the discharge of rain water. The alterations were thus confined to parts which were small, and easily handled, and by this convenience compensated for the greater number of parts. In some of the examples given further on, we shall find the several portions cast in one piece, though larger and heavier than these.

The above descriptions have been chiefly taken from Mr. Digby Wyatt's paper on the "Construction of the Building," &c., &c., printed in the "Minutes of Proceedings, Inst. Civ. Engineers," vol. x., Session 1850-51; and from the work of Messrs. Charles Downes, and Charles Cowper, published in quarto by John Weale, London, 1852. We may also refer the student to the introduction of the "Practical Mechanic's Journal—Record of the Great Exhibition of 1862," by Mr. R. Mallet, in which will be found an instructive account not only of the building of that year, but of all the great Palaces of Industry which preceded it, including this of 1851.

186. The well-designed warehouse, or goods store, forming part of the Midland Great Western of Ireland Railway Waterside Terminus, at the North Wall Quay, Dublin, affords an excellent example of good engineering in the gene-

ral design, and in the detail of the columns and girders of the interior of the building.

It has at the present day become almost a necessity for every railway carrying goods or minerals, whether imported or destined for exportation, to make such arrangements as that the waggon and the ship can be brought so close to each other that a revolving crane may be able to transfer the load from the floor of the former to the hold of the vessel, without any intermediate carrying or cartage. For merchandise of a perishable character a warehouse at the river side, or the terminus, where such goods may be stored under cover from the weather, and into which the railway waggon, on central lines of railroad, and parcel carts, by suitable side openings, may enter, also under shelter, is no less urgently required. All these essentials were amply provided by the Company at this building.

187. The transverse section and plan, Plate VIII., figs. 57 and 58, show that four lines of railway enter the building; and that every waggon, wherever standing, may be loaded or unloaded by cranes, without that troublesome shifting of those contiguous to it which is necessary with other and inferior arrangements.

It will be perceived that there are three classes of columns, marked with the letters A, B, and C. Those marked A support each four girders at right angles to each other, and are not continued upwards,—sustaining, therefore, but the weight of a certain part of the flooring and the goods stored on it, computed at 200 tons. Those marked B support three girders, attached to a connecting piece and an upper pillar. The columns lettered C sustain nearly half the weight of the roof, which is therefore transmitted downwards to the pillars, B.

Column C was the smallest employed in the building, but the dimensions sufficient for its load would not have been equal to the additional duty of carrying the revolving crane. In all the columns the ultimate strength as pillars was more than four times the utmost load which could, under any circumstances, come upon them.

The drawing, fig. 59, shows an elevation of the column A, at a scale of half an inch to a foot, with the base plate attached; and fig. 60, immediately below, shows a plan of the base plate at a scale of ⅜ths of an inch to a foot.

188. The natural foundation consisted of a bed of sufficiently compact and uniform gravel, on which, underneath the position of each pillar, was built a mass of calp limestone

masonry from eight to ten feet square on the bottom bed—the depth to which it was carried varying with the distance at which the firm gravel was reached through the overlying loose soil and made ground. A square cap of granite, about two feet in thickness, and three to four feet in the side, surmounts the pier of masonry; and its surface having been brought to the proper height, dressed, and levelled, placed in range, and the sides made parallel to the outer walls of the building, four holes were then drilled in each stone, on the lines of its diagonals, and two feet from centre to centre; in these were leaded down four wrought iron vertical bolts, having screwed ends projecting above the stone sufficiently high to pass through the base piece, and afford room above it for the fastening nut.

The base plate, shown in plan in fig. 60, and in vertical section, with the column resting upon it, in fig. 61, was not planed on either the top or bottom bed, but merely cast with great care, to insure parallel and true surfaces. It consisted of a square piece of metal, 2 ft. 6 in. on the side, and 2 inches thick, bevelled off on the upper surface for a breadth of 5 in. on every side, so as to be one inch thick on the outer edges; four bolt holes, *a a*, 2 inches in diameter, and 2 ft. apart centres (coinciding therefore with the screw bolts let into the granite cap, already described), are placed at the angles of the base plate on the line of intersection of the planes of the bevelled surfaces mentioned above; each bolt hole is surrounded with a rib $\frac{3}{4}$th in. thick, and $\frac{1}{4}$th in. high, upon which, as upon a washer, the nut rests when screwed home: by this connexion with the masonry the base plate is both held down, and secured from any lateral shifting. The 5 inches of bevelled surface on each side leave a square of 1 ft. 8 in. on the upper surface of the base plate, in the angles of which are cast four other bolt holes, lettered *b*, 1$\frac{3}{4}$ inches in diameter, and 1 ft. 4 inches centres, to receive the dowel pins, which, projecting above the base plate, pass up through corresponding holes, also lettered *b*, in fig. 62, left in the square flange, which, it will be perceived, is cast in one with the shaft of the pillar; these dowels are run with cement, and thus secure the pillar from shifting laterally on the base plate below when once truly placed on it. A rib, $\frac{1}{2}$ inch in thickness, and $\frac{1}{2}$ inch high, shown in plan at *c c*, fig. 60, and in vertical section, fig. 61, forming a square, 8 inches in the side, rises above the upper surface of the base plate, with the edges of which it is parallel, and from which it is 8 inches distant on every side. The base plate, being dropped over the screwed ends of the projecting bolts, is very truly bedded in Portland cement or fine mortar on the upper bed of the granite stone; and being carefully levelled, the nuts are then tightened home on the screw bolts and raised ribs, cement being filled into any spaces which may be found between the bolts and the holes in the base plate.

189. The total height of the shaft of the pillars, A, A, is 19 ft. 9½ inches: its external diameter at the bottom is 1 ft. 6 inches, and at the top 1 ft. 4 inches, giving a taper of about 1 in 234. The thickness of the metal is cast at 1⅛ inch throughout all parts of its height.

On each end of the pillar square flanges, shown at *e e*, are formed and cast in one piece with the hollow shaft, that at the bottom being 1 ft. 8 inches in the side (the same as the upper flat surface of the base plate, as already mentioned), and 1¾ inch thick, that at the top having 1 ft. 9 inches in the side (to receive the bottom flanges of the beams it supports), and is 1½ inch thick; by this square form of the ends of the pillar, space is obtained at the angles (between that external figure and the inner circular form of the pillar) for the holes through which the several dowel pins, screw bolts, &c., pass.

In order to extend the bearing surface of the columns still further, there was applied, both at the top and bottom, a circular filling piece, fitted in, and secured in its place by cast iron cement, previous to the column being reared into its position. This piece is shown in its position, as fitted in the column, in figs. 61, 62, 63, and 66, and also, separately, fig. *66a*. Its depth was 4 inches, the thickness of the circular plate being ⅝ths in. in that placed at the top, and ¾ths in. in that at the bottom of the pillar, and strengthened by ribs crossing at right angles through the centre, every dimension being figured on the drawings. With the view of retaining these circular plates in position, and enabling the shaft of the pillar the better to receive the load placed upon it above, and again to transmit it to the base plate below, there are cast on the inner surface of the column fillets, or rings, which act as shelf pieces, the lowermost projecting 1 inch, and being 1¾ inch deep, the upper projecting ¾ inch and 1¼ inch deep. This excellent arrangement increases the bearing of the pillar on the base plate by nearly 180 square inches, or about 80 per cent.; the area of the cap at the top of the pillar for receiving the girders being augmented in like proportion.

These two plates were fitted in and secured to the column by cast iron cement before it was raised into position, an annular space of ⅜ths of an inch being left for caulking the cement on the lower plate, and ¼th of an inch on the upper.

This novel mode of construction not only enlarged the bearing area, as already described, but enabled the constructors to dispense with all turning and facing of the surfaces of contact between the shaft and the filling piece, which, with pillars of such weight and dimensions, would have been an expensive process. The several pillars, with their circular filling pieces, above and below, duly secured, were raised into position, the four holes in the lower flange being brought immediately over those inner four cast in the base piece, as already described; they were then set truly vertical by temporary iron wedges driven in until an interval of about ½ inch was obtained between the two surfaces; this space was then caulked in with cast iron cement, forming what is called a rust joint—the dowel pins, formed of round iron bar, being dropped into the holes, and cemented up with sulphur and fine sand. The utility of that inner square raised rib, shown in figs. 60 and 61, and already described, will now be apparent, as it afforded a firm point of resistance to the caulking tool when forcing in the iron cement, which would otherwise be driven out on the opposite side, an abutment of some sort being always required in this operation. As the work proceeds, the temporary wedges are withdrawn, and the cement trimmed off in line with the square flange of the pillar.

This valuable material, which consists of less than one per cent. of sal ammoniac—the exact percentage would seem to be unimportant, and is very variously stated—mixed with finely comminuted turnings or borings of cast iron, and sufficient water—has long been in use in fitting up the different parts of bed plates, and other heavy castings; but it is believed to have been first applied in this structure to the erecting of pillars; it evidently acts in the same manner as a bed of mortar between two ashlar stones, which, however truly dressed, would, if laid dry, have but comparatively few points of contact; the iron cement, in the same manner, in separating the two surfaces, adheres firmly to them both, taking also a perfect seal or impression of the minute irregularities of each, and by its gradual hardening is thus enabled to receive and transmit the pressure uniformly over all the points of the surfaces, whether recessed or raised. We have, in the beginning of this work, seen the advantage which Mr. E. Hodgkinson obtained by applying thin sheet lead in specimens under compression (p. 69), and doubtless the advantage derived in any materials from interposed beds of cement may be explained in the same manner.

190. The plan of the top flange of the pillar, with the circular filling piece fitted in, is shown in fig. 66, and also the position of the eight bolt holes, 1½ inch diameter, for the screw bolts that hold down and secure, laterally and longitudinally, the four conterminous girders which meet over pillar A, each girder having two screw bolts through its bottom flange. In fig. 65 is also shown a plan of the top, and a horizontal section of the four beams through the line F, G, on fig. 63, with the screw bolts holding them down to the pillar. Had the wrought iron girders been placed directly upon the top of the pillar, the contact would only have been at a few points of bearing, namely, at the several rivet heads, which could not well have been countersunk so as to give a uniform flat surface. The same cement used at the fitting up of the base of the column was applied here also with great advantage. It will be observed that the girders are of different strength, and slightly different depth, having been designed according to their various computed loads, the strongest having at the pillar ⅜th inch and ½ inch plates riveted together for the bottom flange. The lowermost of these plates, in the deepest and strongest girder, is carried full across the cap of the pillar underneath the bottom of all the other beams, as seen in figs. 63 and 64. When laid in position, this beam was supported by temporary iron wedges at an interval of ½ inch above the pillar; and then a packing piece of cast iron, 3 inches wide, 1⅜ inch thick, and 1 ft. 9 inches long, having two bolt holes corresponding with those in the lower flange of the beam and cap of the pillar, was placed on each side; the next operation was to join together and rivet up the three other girders, and screw down their lower flanges to the pillar; finally, a caulking of iron cement is driven in, which, taking the exact form of the bottom plate of the lower flange of the larger beam with its rivet heads, and also that of the surface of the top of the pillar, gave, when it had set, as full and perfect a bearing of the beam upon the pillar as could be obtained if the two surfaces had been planed and fitted. The cast iron filling pieces, it is evident, act not only to retain the cement during the caulking, but also give a fuller bearing to the two girders which are placed immediately over them.

Those pillars which carry a second range vertically above them, as B, B, &c., have a connecting piece cast upon them, with four vertical faces ranging with four main walls of the building; openings for horizontal bolts are left in them when casting, corresponding to each other on the opposite faces. The vertical ends of the wrought iron girders meeting in line at a pillar are, through these bolt holes, fastened to one another and to the column, by suitable screw bolts.

191. Proceeding now to the examination of the remaining columns, lettered B and C, which are placed vertically over each other, the base of C resting upon the upper flange of B, it is evident, on inspection of the transverse section, fig. 57, that these combined columns perform the duty of supporting a great part of the weight of the roof, together with some part of that of the gallery floor and the goods stored upon it; they also serve to discharge the larger portion of the rain-fall from the roof through the continuous interior hollow pipe which they supply. The column C is also provided, as was that lettered A, with turned external flanges for a revolving crane, the fall of which drops vertically over the centre of any waggon standing on the line of rails immediately below, as shown in fig. 57.

The columns B—constructed, as to their base pieces and mode of fixing, in a manner precisely similar to that already described for column A, and so far needing no further mention here—are shown in general elevation, at a scale of ½ inch to a foot, in fig. 67; and as to the detail of the upper part, in figs. 68 to 73, at ¾ths of an inch to a foot. It will be observed, however, that, in the details of its base, the arrangement as to the final discharge of the rain-water differs advantageously from the usual practice in structures similar to this; in which generally we find that the upper surface of the stone base, and consequently the bottom of the cast iron base plate, is set on the level of the intended surface of the ground. In order, therefore, not to interfere with the clear space around the bottom of the column, the rain descending from the roof is necessarily discharged through a circular opening cut vertically through the stone base in continuation of the interior hollow of the columns, the base plate itself being also necessarily open in the centre. Now, such an opening cut in the stone base diminishes its strength, and exposes it to the risk of being split vertically when the full load finally comes upon it, and thus seriously endangers the whole structure. This system of construction is, moreover, incompatible with the advantages to be obtained from the use of the filling pieces described above. The rain water, also, unavoidably enters the masonry, more or less, and tends to injure it, and render damp the adjacent ground. It is true that a column with lateral discharge pipe must be founded about a foot or eighteen inches lower than when the other method of constructing the base is adopted, in order to place the lateral pipe completely below the intended ground line, and obviate any interference with the clear available space around the column, and its total length is consequently increased by the same amount; but neither this nor the increased quantity of metal which the columns demand from their greater length counteract the advantages which follow its adoption.

192. The diameter of column B is, at the base 1′ 6″, and at the top 1′ 4″, the thickness being one inch at every part. The connecting piece, which is in one casting with the shaft of the column, consists of a bottom flange of a square form 1′ 9″ in the side, and 1½ inch thick, strengthened by two brackets underneath on each of those two opposite sides, which carry longitudinal girders, and but one only on the remaining faces, as shown in fig. 69. The upper flange is also of a square form, 1′ 5″ in the side, and 1¼″ inch thick; and between these two flanges is placed a short pillar of a square form in the horizontal section, 12″ in the side, and 2′ 4″ in height, with a thickness of one inch. Central openings are left in both of these flanges when casting, affording a continuous passage for the descent of the rain water, which is received from the superposed column C, and discharged through the internal hollow connecting piece and shaft of column B.

The relative dimensions of the lower flange and square pillar of the connecting piece give a width in the horizontal direction of 4½ inches on every side, which on three of them is employed to support the girders and contain the screw bolts which hold them down, as shown in the two elevations, figs. 69 and 70, and the horizontal and vertical sections, figs. 72 and 73.

The main or longitudinal girders have their upper flanges bolted together by three horizontal screw bolts, 1¼ inch diameter, passing through the connecting piece, and through a plate of wrought iron riveted on the top flange of the girders, and turned up at right angles to coincide with the flat faces of the pillar, as will be understood from the drawings, figs. 69, 70, 73; two vertical screw bolts also secure the lower flanges of the girders severally to the bottom flange of the connecting piece C. The transverse girder is held by four screw bolts, 1¼ inch diameter, differently arranged from those described above for the longitudinal beams, to suit its lighter character, and as having the opposite face of the connecting piece unoccupied; for the bolts pass through the vertical end plate at about one-fourth of its depth, measured from the top and from the bottom flange, respectively. Two smaller screw bolts, ⅞th inch in diameter, also pass through the lower flange of the girder and of the bottom flange of the connecting piece described above. These firm connexions of the bearing ends of the girders give them great additional strength, rendering them analogous to those which are built into their abutments, or to such as are carried over successive piers in one continuous line of girders, and not merely laid singly upon their bearings.

On the side of the connecting piece, towards the open central part of the building, its upper flange, with a width of 8 inches, projects outwards for 15″, and is supported underneath by a bracket of a depth equal to that of the connecting piece itself, as shown in the drawing, fig. 73. The object of this bracket was to give part of the support to a longitudinal railway bar (one on each side of the main central opening) on which travels a steam hoist commanding every waggon standing on the two central lines of railroad below, and also working wall cranes to discharge or raise goods at the side windows of the upper floor, as shown in fig. 57.

193. The column lettered C is shown in elevation in fig. 75 at a scale of ½ inch to a foot, and with its details in part drawings at ¾ inch to a foot in fig. 74, which is a horizontal section through *n.n.* figs. 73, 76, 77, 78. It has a total height of 11′. 1½″, being 12 inches in diameter at the base and 11″ at the upper end, and 1 inch in thickness. The bottom flange is bolted down on the top of column B by four screw bolts on two of its opposite sides, as shown in figs. 69 and 74. The inner circle in fig. 74 represents the opening for the discharge of rain water; and the vertical section of column B, fig. 73, shows the circular fitting strip cast below the bottom of the flange of column C, with the space for the rust joint, which is made good up to the outer edge of the square flange.

The connecting piece of this column, which is in one casting with the shaft, will be readily understood from the drawings, fig. 76 to 78: the vertical square pillar between the flanges is 9 inches on the side, and has cast upon it a quadrant pipe, opening upwards, intended to receive the rain from the roof, 6 inches in internal diameter, with a socket at the end in order to admit a vertical pipe, which is cast with a double bend, to enable it to be led past the longitudinal girders attached, and brought nearly centrally underneath a box, open at top, and about 4 inches deep, placed immediately below the level of the valley of the roof; the rain is made to flow towards this point by giving the valley the usual fall of about 1 inch in 10 feet from the middle point between the successive pairs of columns; thus each of the hollow pillars has a well-defined and equal portion of the roof area to drain. The box above mentioned is lined with sheet lead in continuation with that of the valley. The angle irons of the bottom flanges of the upper wrought iron girders, supporting the roof, which are of the lattice form of construction, are bolted down on the bottom flange of the connecting piece by inch screw bolts. The top flange of the girder is given all the advantages of a fixed and continuous beam by a very ingenious arrangement: the total depth of the girder, being 2 ft. 1¼ inch, gives it a height of 8¾ inches above the extreme top of the column C; and a casting in two pieces (somewhat similar to the cast iron chairs which hold in place some of the forms of rails now in use on railways) embraces the top flange of the girder on each side, and the three are united together by an inch screw bolt passing transversely through all, as shown in figs. 76, 77. A suitable form is given to the inner surface of the castings, adapting them to fit the angle irons and rivet heads of the upper flange of the wrought iron girder; three vertical screw bolts on each side fasten these castings down on the top of the upper flange of the column. The object of the double bend in the connecting rain pipe, mentioned above, is evidently to bring it clear of this girder, and at the same time nearly underneath the centre of the valley of the roof, which is constructed of timber, and covered with slates and glass; the tie beam of the principal, 12 inches deep by 8 inches wide, rests in a wrought iron shoe, 3 ft. long, which contains the scarf, and is shown in figs. 76 and 77, bearing transversely upon the top of the upper flange of the lattice girder.

194. It will be perceived from the plan and transverse section, figs. 57 and 58, that there are four longitudinal ranges of columns within the building, supporting galleries and the roof; the two nearest the central line are those already described under letter B, and the two outermost under letter A.

The total length of the building, inside measurement, is 339 ft.; and the width, inside the projecting pilasters, 127 ft. 9 inches. The width between the two central ranges of columns is 25 ft. 9 inches from centre to centre, or 24 ft. 3 inches in the clear; after deducting 18″, that is, one diameter. Between these and the ranges, letter A, on each side, the width, centres, is 19 ft. 3 inches; and from the centre of these, again, to the pilasters of the side walls, 31 ft. 9 inches. In the direction of the length of the building the pillars are all 31 ft. apart, centres; and the last at each end are spaced 30 ft. from the inner face of the end wall, making the clear span in every opening about 29 ft. 6 inches.

It is evidently important to place the ranges of columns B as close together as the double line of way between them and the circulation of the goods waggons will admit of; for this dimension rules the width of the central opening extending up to the roof, and consequently limits the available breadth of the storage room on the galleries above, and the space for receiving and delivering goods, &c., on the ground floor below.

Lines of railway are always given 6 ft. in the clear between the up and down lines; adding to this the width of the top of the rails, *i. e.*, 2½ inches, taken twice, and the width of gauge, which in Ireland is 5 ft. 3 inches, we have for the distance from centre to centre of the two lines of way 11 ft. 8 inches. But, as within the building there are three transverse ranges of turntables required to facilitate the shifting of the waggons from one line to another, it will be evident that the least distance from centre to centre of these turntables limits the central distance of the lines of railway on which they are placed. Now, as the turntables are 12 ft. in diameter in the moving part, and 12 ft. 8 inches to the outside of their containing circular frames, it follows that when these are placed in contact we shall have their distance from centre to centre, and consequently that of the two middle lines of way also, 12 ft. 8 inches; the outside width of the covered waggons being 8 ft. 7 inches, we shall have, by deducting the sum of these two (*i. e.*, 12 ft. 8″ + 8 ft. 7″), or 21 ft. 3 inches, from the clear width at a pair of columns, which, as already mentioned, is 24 ft. 3 inches, a remaining space of 18 inches in the clear when a waggon passes a column, which may be considered as just sufficient for safety and convenience.

Outside the range of columns B, on each side of the centre line, there is a width of 11 ft. 3 inches in the clear, adapted for one line of rails; the three-ton revolving cranes on columns A command the centre of any waggons standing on these lines, and, having a radius of 12 ft. 6″, can deliver goods into carts within the building, and under cover, the spaces for which and gateways through the side walls are shown in the plan, fig. 57.

195. In order to estimate the greatest load which can by possibility come upon the several pillars, we must first study the arrangement adopted for the main beams and timber joists; and, secondly, find the maximum load which can be placed upon the galleries; and, lastly, compute the weight of the main girders and flooring, the roof, and that of column C, with its revolving crane, and also the weight of the steam hoist, which, travelling the whole length of the building, may at any time be working over one of the columns B. Wrought iron beams are laid transversely from the side walls to each column A, being ten in number under each side gallery; in the longitudinal direction four lines of beams are laid from each end wall to the successive contiguous columns in the several lines of A and of B, and longitudinal beams are also fixed, one under each gallery, between the middle points of the transverse girders mentioned above, from end to end of the building; all these six lines of beams are shown in transverse section in fig. 57, and are eleven in number of each class on either side of the building. The clear span of each of these several classes of girders is very nearly the same, being about 29 ft. 6 inches for the longitudinal, and 31 ft. for the transverse openings; the girder connecting A and B in the transverse direction is only a tie to steady the head of column B, and is not properly acting as a beam, but merely retaining it from any lateral motion The timber joists are all placed in the transverse direction of the building, and rest upon sleepers 12″ deep by 4″ broad, laid on the bottom flanges of the beams, and bolted together by screw bolts passing through their vertical webs.

196. Considering the nature of the goods' traffic on this line, it is probable that the maximum load which can be placed on the galleries would be that of corn, which if in sacks could be heaped up nearly vertically on the open side of the central opening. We may assume the weight of corn on an average to be 60 lbs. per bushel, which, as a measure of capacity, is defined by statute as containing 8 gallons of water, or 80 lbs. weight; we shall have, consequently, a uniformly distributed load of corn in sacks, at the above density, equal to ¾ths of the weight of water of the same depth. Let us suppose the floor to be loaded to a uniform depth of 12 ft., or nearly up to the level of the tie beams, and we shall have, from the

above figures, the pressure equal to ¼th ton per square foot. Now, as to the several columns A, the area of gallery floor which may be taken as resting more particularly on each is 720 square feet, or a rectangle whose sides are found by taking half the distance to the side walls and to the range of B for one side, and half the distance to each of the contiguous pillars A for the other, as shown by the dotted lines on part of the general plan, fig. 57: the weight to be supported by each pillar A will thus be 197.5 tons, and the girders and flooring at about 30 lbs. per square foot, equal to 10.58 tons, altogether about 208 tons. In like manner about 310 square feet, with the same weights per square foot, will be thrown on each pillar B, or 77.5 tons, and the gallery floor, as for A, equal to 4.15 tons—total, 81.65. The side walls, again, for each length of 31 ft. will support about 496 square feet, equivalent to 124 tons. The columns B also support C and its proportion of the upper longitudinal girder under the valley of the roof, together with about 1300 square feet of the roof itself (measured on the sloping surface) which at ¼ cwt. per square foot gives 16.2 tons; the columns C, with the longitudinal lattice girder under roof, amount to about 1¾ tons; and the revolving crane and its load of 3 tons make together 5 tons, giving a total weight = 21.2 tons. The load on B may also be increased by the travelling steam hoist being placed, when at work, immediately over a pair of these pillars on opposite sides of the central opening, which we may take as equivalent to about 4 tons on one pillar; and therefore the extreme weight on one of the pillars in the range B may be taken at 103 tons. It will be readily perceived that, at whatever intermediate point the column A may be placed in the width between the side wall and the corresponding pillar B, no alteration is thereby produced in the load it has to sustain, which is always one-half of that on an area of 51 ft. × 31; the remaining half is divided between the side wall and B proportionally to the segments into which A cuts the width of the gallery, the greater load being towards the greater segment.

197. As it was foreseen that the galleries above would at times be very unequally loaded, and the beams deflect to some extent, so that at the bearing points they would throw an oblique pressure on the top of the columns A, their thickness was consequently increased from 1 inch, which would give a sufficient factor of safety, to 1⅛th inch. By placing the pillars A at about ⅗ths of the width of the ground floor between the side walls and the range of B, the space is conveniently di-

vided; between A and B there is width for a line of rails and landing wharf for goods; and the fall of the revolving crane, having 12 ft. 6″ radius of sweep, will on one side command the centre of the railway waggon, and on the other that of the cart, which with the horse attached can enter, completely under cover, through the gateways opened in the side walls.

The Author is indebted for the drawings and details of this structure to the kindness of Mr. G. W. Hemans, Engineer-in-Chief; and to Mr. W. Anderson, at that time of the firm of Messrs. Courtney, Stephens, and Co., who constructed the iron work of this building.

198. In the two following examples we have instructive instances of the first of the different divisions into which structures supported on cast iron pillars have been conventionally classified—*vide* p. 170, § 169; namely, the Carlisle Pier, situated within the Kingstown Harbour; and a landing shed on the North Quay at Dublin, which, as we shall see further on, are both necessarily open on all sides, in compliance with the objects for which they were erected.

When the determination was formed to accelerate the mail service between London and Dublin, so as to occupy only eleven hours in the transit between the two cities, it was immediately seen that a landing place exclusively appropriated to the new steamboats must be constructed within the Kingstown and Holyhead Harbours, so as to obviate all that interruption and loss of time which would otherwise continually arise.

The Carlisle Pier was accordingly run out into the requisite depth of water within the harbour at Kingstown, and over part of it was placed a roof supported on cast iron pillars, with the object of affording shelter both to the passengers and to the agents of the Post Office, and other *employés*, on embarking and disembarking as they passed between the railway train, drawn up under the roof, and the mail steamboats alongside.

199. The position of the pier within the harbour was chosen after considerable discussion, and no small difference of opinion as to the best point for its location. Several conditions were obligatory in this selection; and, to understand them more fully, it may be necessary to state some of the physical features of this harbour, which is shown in plan in fig. 80, Plate IX., at a scale of 6 inches to a mile. The rise and fall of the tide at full and change of the moon, that is, at spring tides, on this part of the coast, is about 12 ft. or 13 ft.,

and the depth of water at the entrance of the harbour about 25 ft. at low water, shoaling off towards the line of the shore, but leaving a large part of the 270 acres which the piers inclose with a depth of about 20 ft. The prevailing winds, which come from the west, are off shore, and the harbour is landlocked at all points except from about north to east, so that in gales from this quarter the inner western side of the harbour is rather exposed, and subject to a heavy swell.

The mail boats, which have a speed of about 16 or 17 miles per hour, are 300 feet in length, and draw from 10 to 12 ft. Now, the establishment of this port, that is, the hour at which high water of spring tides occurs, is eleven o'clock in the forenoon; and these boats, being timed to arrive and depart at about six and seven o'clock both morning and evening, are thus generally alongside, under steam, at an hour not far removed from the low water of springs.

From these different considerations it was evidently essential to select for the site of the pier some part of the harbour where, in the first place, there would be deep water alongside; and, secondly, the least possible exposure to the effects of gales of wind from the north round to east, and moreover to secure these advantages without interfering with the deep-water moorings and room for tacking already provided within the harbour. It was also necessary, in making this choice of site, to keep in view the possibility of bringing a branch line from the Kingstown Railway down the centre line of the intended pier. All these conditions, it seems, were best fulfilled in the site ultimately chosen. Its direction, pointing towards the entrance of the harbour, enables the mail boats to come alongside and start without delay. The depth of water is ample at all times; and from its almost recessed position in the south-eastern angle of the harbour, no interference with the anchorage results; and in gales from the north-east the water is less disturbed than at any other place which might have been selected. In order to lessen the disadvantages which boats and small craft would otherwise suffer, an opening has been left in the pier near the shore, and arched over, so that there is a communication at all times underneath. However, to reach the pier by the branch from the main line of railway to Dublin, it was necessary to adopt a curve of about 313 ft. radius, that being the very least which this Company's engines could traverse; and even with this radius a large part of the curve had to be laid on the pier itself, with a second bend, or S-curve, extending nearly up to the landing shed before it could be brought in line with the direction of the pier.

The dotted lines in the general map of the harbour, Fig. 80, extending outwards from the Transport Service Wharf, indicate the position for this pier which was advocated by the late Mr. Rendel.

200. The requirements of the Postal Service, and the nature of the accommodation necessary for passengers, indicated pretty clearly the general dimensions and details suitable in the design for the pier, and for the roof, &c., of the shed to be erected upon it, with its supporting pillars.

Thus we see that, as a consequence of the near coincidence of the appointed hours of arrival and departure of the steamboats at the Kingstown side, both with the morning and evening mails, it became necessary to appoint one side of the pier and of the shed (the western) for the use of the boat arriving from Holyhead, and the other (the eastern) to that leaving Kingstown for the former place. It is evident, therefore, that each side of the shed must be alike completely open, so that the construction of any wall of masonry, of which advantage might be taken at once to support and also hold down the roof, is inadmissible; and thus the stability and permanence of the structure in the gales of wind to which, from its situation, it is so much exposed, must depend solely on the secure anchoring down of the cast iron pillars at their bases, and the firm attachment of the roof principals to the connecting pieces above. We shall find that the pillars, &c., have been carefully designed in anticipation of these several requirements; and it may be added that, for the same reasons, connected with the hours of the mail service, a double line of railway for the carriages attending on each of the steamboats must be provided under the cover of the landing shed. The gable ends of the roof were also necessarily open—the southern, to admit the entrance of the engines and carriages from the branch line of railway; and the northern, in order to give access to the turntable placed outside at the extreme point of the pier, as shown in Fig. 82.

201. The different dimensions of the pier and roof are figured on the transverse section, Fig. 81. The two ranges of columns which support and steady the roof are, it will be seen, placed 34 ft. apart, transversely from centre to centre; of this width 24 ft. are allotted to the double line of railway, thus leaving 5 ft. on each side for the convenient movement of the passengers entering or leaving the carriages. The level

of the line of rails is about 2 ft. below the platform; and as the newer carriages on this line are about 10 ft. wide, measured at the steps, there is, when trains are standing on each line of railway, a clear intermediate space of 4 ft. in the centre, which is required by the *employes* of the railway company in their various duties. The pent roof or hanging shed extends, including the rain gutters, 11 ft. on each side beyond the range of columns, so that the total width of covered space is 56 ft. In each line of columns (Fig. 82) there are 15 pillars, placed 18 ft. apart, centres; the extreme length of the roof being 254 ft., which gives room for about six or seven carriages to stand under cover on each line of rails; the total horizontal area covered over being therefore 14,224 square feet.

202. The form and dimensions of the pillars are shown in Figs. 82, &c. The connecting piece, shaft, and base piece are all in one casting. The base, which is square in plan, and 2 ft. 3 in. in the side, is let into a massive granite block, so that its upper surface is flush with the level of the platform; each pillar is fastened down by four lewis bolts, leaded down securely into holes prepared in the stone, and having screwed ends 2 in. in diameter, projecting upward, and so spaced as to pass through openings left in each angle of the base, the plate of which is two inches in thickness, except at the several angles, where for a space of 9 in. square it is increased to 4 in., the projection being on the under side, as shown in Figs. 89 to 92; the granite stone being carved out with a corresponding recess. This additional thickness at the angles is given to the base piece to enable it fully to receive the holding down hexagonal nuts when screwed home on the upper end of the lewis bolts, and consequently leave their upper surfaces on one and the same level as the base of the column and the platform; a circular recess was therefore cast on the upper surface, at each angle of the base piece, of sufficient diameter to enable a spanner to work the nut, and of a depth sufficient to contain it when home, which end could not have been effected unless by this increased thickness of the base plate at the angles.

The diameter of the pillars at 3 in. above the ground line is 10 in., and the thickness 1⅜th in.; at the neck these dimensions are 8 in. and 1⅛ in. respectively, so that the external line of the column tapers about one in a hundred on each side. Between the surface of the base and the above-mentioned height of 3 in., the form is a quadrant of a circle, with a radius of 3 in., forming a cavetto or hollow moulding. Above the neck of the pillar a sort of ogee moulding projects for the purpose of carrying the octagonal head.

The connecting piece, or head of the pillar (figs. 84, 87, and 88), is externally octagonal in form, and bored out so as to receive accurately the cylindrical turned foot of the cast iron standard. These standards are shown in Figs. 85, 86, and, being let down into the head of the pillar, are secured by means of a wrought iron key passed through openings left in the pillar head and standard, diametrically opposite to each other, and firmly keyed up, as shown in the drawings. The rain water is received in galvanized wrought iron gutters, and carried by pipes of the same material into the alternate pillar heads, and thence through the pillars into drains prepared below. The disadvantages of discharging the rain water through the base of the pillars, already dwelt upon, do not apply to columns such as these, which, as not bearing great insistent load, do not require an extended area to rest upon; and, for the same reason, the massive granite blocks are practically uninjured by the central hole drilled through them.

203. The pillars, with their standards in place, having been erected truly vertical, in the same horizontal plane, and parallel and perpendicular as regarded themselves and the axis or central line of the building, and being securely fastened at the base, as already described, to the prepared granite blocks, and at the given distances, were ready to receive the main tranverse principals and longitudinal girders or purlins, which, though constituting with the roof covering the actual and only vertical load that the pillars have to sustain, yet aid them materially in performing their chief duty as unyielding vertical beams, by forming one complete frame tied in every direction, so that any lateral force of the wind acting upon the roof is resisted simultaneously by the strength of all the columns, thirty in number; the main and intermediate principals, and lower and upper longitudinal girders or purlins, being riveted to each other, and to the standards, and stiffened by diagonal straps of wrought iron, $2\frac{1}{2}$ in. by $\frac{1}{4}$ in., passing from the heads of the standards on one side to the alternate opposite heads, and securely fastened to them and to the upper flanges of the principals they severally pass.

It will be evident that, in sustaining the weight of this roof, the pillars exert but a very small fraction of their strength as columns. To calculate that strength we should take, as being on the safe side, the lesser diameter at the neck of the pillar, i.e., 8 in. for the external, and $5\frac{3}{4}$ for the internal; and as to the length, we may assume it as being equal to 10 ft.; the vertical load on each cannot be estimated to exceed 7 tons. The true duty of such pillars is, as has been mentioned, to act as vertical beams fixed at one end, that is, the base, and pressed at the other by the lateral or horizontal force of the wind impinging on the roof. The effective area acted upon by the wind will evidently be equal to the product of the total length of the shed into the vertical projection of the sloping side, that is, 254 ft. into 12 ft., which gives 3048 square ft. Now, the extreme pressure of the wind may be taken at 55 lbs. per square ft. in the highest gales in this climate, giving a horizontal force of 74.8 tons, or about 2½ tons to each pillar; and as the roof is a rigid frame, and immovably united to the pillars, the leverage of this force to fracture them transversely at the base must be measured by the vertical distance from the middle point of the rise of the roof above the ground line, which is about 16 ft. The experiments and resulting formulæ and constants for hollow cylindrical cast iron beams, which will be given in a future part of this work, indicate an ultimate transverse strength in these pillars about four or five times in excess of the above strain.

204. The details of the pillars in this structure seem well worthy of adoption in similar circumstances. The base piece, for instance, with its holding down nut and screw bolt, being made flush with the surface of the platform, offer no impediment to the generally hurried movements of the passengers or porters at the arrival and departure of the trains. It is matter of surprise how often this obvious detail of pillar construction is neglected; the edge of the cast iron base being often placed for its full thickness above the platform level, and moreover carrying on its upper surface the projecting holding down nuts, with an inch or more of superfluous screw bolt above it, all tending to trip up the unwary crowd.

The curved form of the bottom of the shaft of the pillar is better practice than bringing down the exterior line of the shaft to intersect at an angle with the horizontal surface of the base, and supplying the needful stiffness of the base by inserting a number of radial gussets, not only from the number of re-entrant angles, which endanger the soundness of the casting, but also from the greater interference with the space around the foot of the pillar than in this example. All the points we have mentioned in contrast are shown in figs. 92*a* and 92*b*, which are taken from existing structures.

The increase of the diameter from 8 inches at the neck to 10 inches at the base, and of thickness from 1⅛ to 1⅜ inches at the same parts is judicious and economical, remembering that the pillar is to be regarded as a vertical beam, and that the maximum bending moment is at the base. It has been proved experimentally by Mr. Hodgkinson, that in pillars carrying vertical load only, no practical advantage is gained by adopting a tapered instead of a true cylindrical form.

The method of securing the standards by inserting them into hollow caps or heads prepared in the pillar obtains great favour amongst the constructors of wrought iron roofs supported on pillars, and certainly affords a good resistance to a horizontal force, when carefully fitted. It was intended to have had an internal stud in the pillar head and a returned slot in the standard, but all such studs are uncertain in a material like cast iron, and liable to be unsound with the utmost care in the foundry. Keys driven through well-prepared keyways or mortice holes in both the head and the standard can be made to fulfil every required end.

The double T-form of the upper part of the standard is well adapted for joining on to the wrought iron roof frame; the heel of the main transverse principal on each side of the shed being bolted on at the side lettered *a* by ⅝-inch screw bolts and nuts, and the head of the hanging shed which projects outside the line of pillars on the other; and they are all three further secured by a wrought iron strap passing over the standard head, and fastened to the upper flanges by rivets.

The main or lower longitudinal girders are also secured to the standard head on the two other and opposite faces by screw bolts and nuts; the upper girders or purlins, parallel to these, are rivetted to the main transverse principals, two intermediate principals rest on the girders, and are framed and secured to them, the whole being stiffened by the diagonal ties already mentioned. In connexion with the detail of the standards it may be pointed out that the spacing of the pillars, 34 ft. apart transversely, and adopting a hanging shed for 11 ft., externally, on each side, has the advantage of affording a sufficiently deep and secure junction between them and the principals, and removing the lines of pillars further from the water's edge, thus giving uninterrupted space for the work of mooring the steamers as they arrive, and also room for the requisite bollards. These important points would not have been attained had the lines of pillars been placed 54 ft. apart, and a trussed principal of a span thus increased by 20 ft. attached only at the cap of the column, which would have been the case, had the pitch of the roof and height from the platform to the rain gutters been maintained the same as in the existing roof.

205. The pier was constructed up to the level of low water line by the diving bell, worked from a stage on piles; from this level it was built by tide work, and set in cement. Two massive walls 20 ft. wide were carried up on each side; in the intervening space unsquared masses of granite were tipped in from waggons worked on a temporary tramroad. The centre line of the ranges of columns falls about the centre of the side walls, in which no subsidence or derangement need be feared, and the ballast for the railway has been laid on the looser central filling. The work has proved very satisfactory, no vibration being apparent even in the heaviest gales of wind.

The working drawings and specification of the above instructive example have been very kindly lent for this work by Colonel M'Kerlie, R. E., Chairman of the Board of Public Works, by whom the structure was designed.

206. In figs. 93 to 96 are shown the general arrangement and details of the landing sheds constructed on the North Wall Quays, Dublin, by B. B. Stoney, Esq. These structures are of the greatest utility in affording temporary shelter from the weather to merchandise which must otherwise lie exposed until the opportunity for its removal occur; and being most frequently open on all sides, it is necessary that here also, as in the last example, the column be very firmly held down at its base, and at its head be well united to the roof. The foundation, which also is made to answer as an anchorage for the whole shed, consists of a mass of concrete under each pillar of about 18 cubic ft. The excavation for it having been carried down to the proper depth, the first operation must be to place the wrought iron washer plates, about 8 inches square, at the bottom; the holding down bolts, which are 1¼ inch in diameter, having been already passed through them and standing up in the angles of a square 14 inches in the side.

207. The column, fig. 93, is in one casting with its base and cap or head, and, as shown in plan and horizontal section, fig. 94, is of a square form, 10 inches in the side, externally, and 1 inch thick; the base extends 4 inches on all sides, and is consequently 18 inches square, having two gussets at each angle, which unite it strongly to the column; in the square spaces they enclose bolt holes are cast, 1⅜ inch in diameter, and 14 inches centres with raised edges ½ inch in width, through which the screwed ends of the holding down bolts al-

ready mentioned are intended to project upwards. The concrete having been filled in around the bolts, and a thin bed of cement spread on its upper surface at the proper level, the column is let down over the bolts, and firmly fastened by the nuts being brought home on the raised edges.

The cap of the column is adapted by its form to receive securely a longitudinal beam of timber, fig. 95, to which the rafters are attached; the rain water is conveyed from the cast iron gutters through a bent pipe to an opening in the front of the pillar, and downwards through the interior, being discharged into a paved channel on the ground surface, by a simple lateral opening, of a circular shape, at the proper level.

The square form has been adopted in the pillars, as giving greater facilities than the circular for the attachment of gates, partitions, &c. The experience of several years has satisfactorily proved that the concrete block forms a sufficient base for these structures, and of its economy when compared with a block of granite of the same dimensions there can be no doubt. At many ports these structures are of much larger dimensions in every way; and, for bonded goods, brick walling is filled in between the pillars, which are of much larger dimensions than those described above.

208. The following example is intended to show the arrangements and dimensions of the pillars, and their adjuncts, adopted in the construction of Fire-proof Buildings.

In warehouses, sugar refineries, and also in mills for various textile fabrics, such as cotton, flax, wool, and silks, fire-proof structures are now deemed indispensable, or at least the materials of which the buildings consist are required to be incombustible, though they are too frequently destroyed by the conflagration of the enormous quantities of inflammable goods stored within them. In such structures cast iron pillars, under great direct compression, form one indispensable element. In plan the main building of such a mill is generally a rectangle, the dimensions of which, when of first-rate size, will be in length from about 200 to 300 ft., and in breadth from about 40 to 60 ft., or even more, and having five or six stories in height; in plate X., fig. 97, is shown a part plan of a building provided with fire-proof flooring; fig. 98 is a vertical section through the line AB in fig. 97, and fig. 99 a part longitudinal section through the line CD on the same plan.

The general arrangement is clearly shown in these drawings: longitudinal ranges of cast iron pillars support at every floor girders which are placed transversely in the building;

between these beams brick segmental arches, with very small rise, are turned, and concrete filled in to a level surface upon them, on which the floor of landings or tiles is laid. Along the length of each successive story are placed one or more ranges of columns, the distance between the ranges being from 14 to 25 ft., and that between the several pillars in each line from 8 to 10 ft., and sometimes even 11 ft.; the span of the segmental arches being determined by the latter interval. The beams of cast iron (and in the more modern practice sometimes of wrought iron), are laid transversely across the width of the building, and rest upon a wall plate on the main longitudinal outside walls, and upon the flange of the cap of the corresponding contiguous pillar; each pillar, therefore, supports the ends of two conterminous beams, which are firmly united around the connecting piece of the pillar, and are placed on the same level, and in the same right line.

209. In fig. 100 are shown two stories of superimposed pillars, and part of a third, in general elevation on a plane parallel to the width of the mill, on the line AB, fig. 97, the brick arches being removed, at a scale of half an inch to a foot; and in fig. 101 a vertical section through the axis of the pillar in a plane at right angles to the former, that is, parallel to the length of the building, the scale being one inch to a foot. Between each successive pair of the parallel beams, as above mentioned, a brick segmental arch is turned, the outward thrust of which is counteracted by wrought iron tie rods, without which the arches might produce a dangerous lateral flexure of the beams, and destroy the gable end walls of the building. The pillars are also bound together, at each floor level, by wrought iron tie rods, parallel to those first mentioned; and, lastly, a level surface is obtained for the floor by filling in the spandrils above the brick arches with a light concrete of lime and ashes, the floor itself being formed with tiles, flagging or landings, set in a bed of mortar on the surface of the concrete.

The several columns of the lower story sustain the weight of a part of the first floor and of the machinery or goods placed on it (the area for each pillar and for the walls being shown by dotted lines, fig. 97) together with the vertical pressure transmitted through the columns of the second story, which in like manner support part of the weight of the floor above them and the bases of the third story pillars, and so on. The successive columns, placed vertically over each other, must therefore be given powers of resistance proportioned to the very different weights they have to sustain.

Thus the load supported by the pillars on any particular story, as arising from the weight of the building itself, depends conjointly upon the average weight per square foot of the fire-proof flooring, upon the breadth between the ranges, that is, on the span of the girders; upon the longitudinal distance of the pillars in each range, that is, on the span of the arches; and upon the number of storys above the pillar under consideration. The weight—including, of course, the weight of the pillars themselves—of the flooring may be taken as between 140 and 160 lbs. per square foot in factories; that of the machinery may be taken at about 100 lbs. per square foot on the average.

210. We may now consider the columns in detail. The base plate, which in these structures is a separate casting, has an area of from 4 to 5 square feet, according to the height and weight of the building, and, being of a square form, is therefore from 2 to 2½ feet in the side. It is shown in vertical section through the centre in fig. 101, and in isometrical projection in fig. 102, the side being 2 feet 3 inches, and the thickness about 2½ inches, the upper surface being bevilled off on all sides. This base piece is supported on a large block of aisler stone, about 3 feet 6 inches in the side, and from 1 foot 3 inches to 2 feet in depth, set upon footings of brickwork, which extend the final bearing surface resting on the natural ground to about 25 square feet, or more, according to the quality of the foundations.

It has been the practice, in some instances, to cut a recess in the upper surface of the stone block to receive the base plate, the bottom of the recess being dressed and levelled to receive it; the object (if any) of this recess, it is presumed, must be to give lateral steadiness during the erection of the pillars, which service no doubt it would perform when flushed up all round with cement; but the practice is certainly objectionable from the greater difficulty of dressing a recessed surface true out of winding, and of setting it truly level, compared with the same processes when we have a free upper bed of the stone unrecessed, and also from the increased trouble the workmen find in fitting the casting—a circumstance which always tends to the work being inefficiently performed. The dotted lines at *ff*, fig. 101, show the form of the upper part of the stone base in section when this recess is sunk in it. The proper course would seem to be, to dress all the upper surface of the block, as shown in the drawings, and to depend for the requisite lateral steadiness upon iron dowel pins let down into the stone, and passing through holes left in the casting near the angles, all being filled up flush with lead or some suitable cement, as described in some previous examples.

The consequences of the base piece not being truly bedded in a recess sunk to receive it in the surface of the stone may be very serious. The unequal pressure thus occasioned has been known, in a mill of many stories in height, and just when the complete structure had brought the final maximum load of the floors on the columns, to split the stone base, causing a subsidence of the superimposed vertical line of pillars, which, however small it may be, endangers the cast iron girders and brick arches depending on them in every story. The remedy applied in the case alluded to was by temporarily relieving the lowermost pillar of its load by screw-jacks, whilst the two parts of the stone, which had opened out below, were brought together and bound round with a strong strap of wrought iron shrunk on.

The upper surface of the base piece is cast with a cross, shown isometrically in fig. 102, and also in the vertical section, fig. 101; the extreme faces of which were dressed so as exactly to correspond in length with the interior diameter of the lowermost pillar, which, when raised into its vertical position, and dropped over this projection, was thereby secured against any lateral shifting upon the base piece; this surface and that of the bottom of the pillar were faced in the lathe, so as to fit with perfect truth. The exact levelling of the base plate, on which operation depends the verticality of all the pillars resting on it, is carefully performed by a spirit level applied to this surface.

211. The shaft of the column is, ascending from the foundation, the next part to claim attention. In the construction of mills the columns seldom exceed 8 inches in diameter, the thickness of the metal being about 1½ or 1⅜ inches in the lowermost story, decreasing gradually to about ⅝ths of an inch at the uppermost: the external form is cylindrical, and the diameter the same in every story, and with every different amount of loading in the various parts of the structure—a circumstance which leads necessarily to the strength being adapted to the duty by a varied internal diameter, and consequent thickness, thus giving the great advantage of uniform distances and spans on external measurements in every story, as already mentioned in the account of the Exhibition Building of 1851.

The elevation and vertical section of the shaft of the column show that a small portion of the lower part was increased in thickness on the external diameter, like a plinth, from about 8 inches to about 10½ inches, the internal diameter remaining constant; this gives great steadiness, and, moreover, the annular surface which finally transmits the pressure to the base is more than doubled, being, with the above dimensions, increased from 30.5 to 67 square inches, and the unit pressure diminished in the inverse proportion of the same numbers, while the quantity of metal in the whole column is scarcely at all increased; this judicious practice we may observe to be frequently followed. In the column shown at fig. 40, we find the annular base of the pillar increased, by the same method, from an external diameter of 14 inches up to 15½, the internal diameter remaining the same, namely, 11½ inches, the consequent increase of bearing being from 50.6 square inches, which it would have otherwise been, up to 84.8 square inches, or 70 per cent. The danger of splitting the column at the ends in handling it, either in the lathe or in the conveyance and subsequent erection, is much diminished; and any decrease of bearing area from air holes, &c., can be better afforded with the arrangement now indicated.

A suitable capital, easily moulded, and in one casting with the circular shaft, terminates this part of the pillar; this capital, widening outwards in the upper part, gives a bracket-like support to the cap, which has to sustain the end of each of the girders which rest upon it. The connecting piece is also cast in one with the pillar below, of which it is in fact merely a prolongation, except in having a socket at the top of somewhat larger diameter, which is intended to carry the lower end of the pillar immediately above.

All the pillars, being externally identical in every respect, and only varying in the internal diameter to give strength in proportion to the load to be supported, are advantageously made in one casting with the connecting piece. The method of tying together the several columns standing in the same longitudinal range in each story is by horizontal tie rods of iron bars, about one inch square.

The drawing, fig. 100, shows a rectangular opening, with a raised flat surface around it, at the letters *a* and *a*, another similar opening being left on the other side diametrically opposite, the several columns having an identical arrangement; and the line joining these openings, being perpendicular to that of the girders, is therefore parallel to the length of the building. Through these openings the tie rods are passed, each rod extending from the outside of one pillar to the outside of that contiguous to it, in the same longitudinal range: each pillar has therefore the ends of two rods attached to it, as shown at *d d*, fig. 101; they are very ingeniously and simply keyed up and fastened by a single iron wedge, also shown in fig. 101. The bars have at each end a hooked termination, the flat inner faces of which rest upon, and are fitted to, the corresponding flat surface around the rectangular openings in the connecting piece.

When the ends of the tie bars have been passed through—and it is obvious that the hook on each must not be longer than the depth of the opening in the pillar through which it must be passed—the hook of the lower bar pointing downwards, and of the upper one in the contrary direction, then the key or wedge, being driven into the vertical space between the bars, forces each into its intended position, from which they cannot be drawn out by a longitudinal pull, without first shearing off the external projections of the hooked ends; and, as the shearing resistance of wrought iron is equal to its direct tensile resistance, the horizontal area of the hooked end should be at least equal to the transverse section of the tie bar.

212. The form of that end of the girders resting on the pillars, and the method of securing them to each other and to the pillars, both longitudinally and transversely, is shown in fig. 101, at the letters *c c*, and in fig. 103, which is a horizontal section through the level of the tie bars. It will be observed that the bearings of the girders are constructed to fit the cylindrical form of the connecting piece, each embracing half its surface, and having half the cap to rest upon, and are by the same arrangement made secure from all lateral derangement; all which will be readily understood by the isometrical drawing, fig. 104, when compared with figs. 100 and 103. These conterminous girders are tied together longitudinally by wrought iron hoops or rings on each side, shrunk on semicircular projecting horns, cast on the ends of the beams at about half their depth, which project horizontally about 4 inches; when the beams are laid in line, the horns on each side coincide.

213. The socket on the top of each column is faced and bored exactly to fit the foot of the pillar immediately above, the annular bearing area of which is increased, and its external form made similar to that of the lowermost story, by

an external fillet or step, acting as a plinth. The socket also secures the superimposed pillars from any lateral motion when once dropped into place.

The segmental arches are made as light as possible; in factories which have only the weight of the machinery to sustain, the rise is from one-twelfth to one-tenth of the span; in warehouses which carry great loads, and the floors of which are exposed to impact from the moving or the falling of heavy bales and packages, piled as high as possible, the proportion is about an eighth. The bricks, which should be of good quality, hard, and well burned, are moulded to suit the curve of the arch, the springers being shaped so as to fit against the cast iron beam. In factories the lower parts of the arch are in several instances 9 inches deep, then 7, and at the key a half brick set on edge, or 4½ inches; but in warehouses the arches are made stronger, with a greater depth of brick. All these details vary according to the circumstances of the particular structure, the nature of the strains, and the uses for which the building is intended. The haunches are filled up with a light concrete of lime and ashes, which is brought to a level surface, ready to receive the floor already described. The structure being identical in every respect between each line of girders, it is evident that the horizontal thrust of the arches is in equilibrium on each side of every girder, when no settlement in any of them has occurred. At each gable end of the building, and on every story, the axes of the segmental arches are placed in the direction of the length of the building, and consequently at right angles to those already described; this is shown in figs. 97 and 99; it will be observed that the tie rods terminate at the last line of transverse beams, the object being to make room for the short longitudinal beams placed with their ends resting on the gable wall and on the flange of the cross beams, which are made stronger for that purpose. These short beams and the segmental arches they carry are considered to form a strong resisting abutment to the thrust of the transverse arches placed from one end of the building to the other.

If any one of these transverse arches should fail, the whole structure would be liable to serious risk, the immediate consequence being that—unless the tie rods sufficed to prevent it—the unsupported thrust of the contiguous arches would break the cast iron girders, which are formed and proportioned to resist a vertical uniform load, but not a lateral or transverse strain in the horizontal plane, and thus from arch to arch the destruction would spread with great rapidity, and that floor be completely destroyed, the probability being that the floor below in some part would be injured by their falling upon it, and the ruin of the lower floor be equally rapid with that above, and from the same causes, and so on to the very lowest—a disaster which has more than once occurred. The simple application of wrought iron tie rods to take the thrust of the arches in a great degree obviates this danger: two series of these rods, in lines parallel to the length of the building, pass through each girder transversely, holes being cast in their vertical webs for this purpose, around which are raised surfaces like those described already on the connecting pieces of the columns; the tie rods are united firmly to each girder, so that each pair of beams, taken in any order, are held together, and the thrust of the arch between them balanced by an equal and opposite force: now, even if one arch should fail, or have to be taken down for repair, no injury can arise during the operation; of course, prudence would dictate in such a case the insertion of sufficient temporary timber props as soon as possible. Fig. 105 is a horizontal section of a beam at the tie rod, showing the arrangement described. In the earlier days of fire-proof mill construction not a few instances of sudden ruin to the structure, and loss of life, are recorded, even though tie bars had been used, which accidents will be described in a future page.

The tie bars, as well as those described already for steadying the pillars, are placed at a level sufficiently high to obviate their interfering with the headway beneath the soffit of the brick arches, the top flange of the beam, when of cast iron, rising towards the centre of its span, with a curvature which enables the holes through the vertical web to be placed sufficiently high to admit of this being easily effected. Fig. 105 shows a horizontal section through the web of the beam at the opening for the tie rods, which are identical with those applied to the pillars, except that the heads of the rods are laid horizontally, in order to interfere less with the brickwork.

The form and strength of the beams, and the interesting comparison as to the relative economy of employing cast or wrought iron in them, will be given in Part II., in which Transverse Strains, with practical examples, are to be treated of; but it may be seen from the drawings that the top of the beam curves downwards to the bearing end, the bottom line being straight; and this affords room for the tie bars of the pillars above to pass clear over the ends of the girders and the wrought iron straps holding them together.

The arrangements for uniting wrought iron beams to the connecting piece, and to each other, are somewhat different in their details from those above described for cast iron. Figs. 106 and 107 show that the connecting piece is of a square form to adapt it to the end of the girder: bolt holes are left in the pillar, and corresponding holes drilled in the vertical end angle irons of the beams, through which screw bolts are passed, thus uniting all three together as firmly as in the former material. In some cases it will be necessary to provide room for the bolt holes in the connecting piece, in the manner shown in fig. 108; the bolt holes in the end of the beams being required to be spaced at a greater distance than the width of the square part of the pillar. The wrought iron beams, unlike those of cast iron, are given the same depth at all points of the span; the tie rods are simple flat bars, riveted to the top of the upper flange of the beam.

In figs. 109, 110, are shown the hollow bricks used by Mr. Fairbairn at the Saltaire Mills, in forming the arches which support the floors. "This adaptation is a great improvement, inasmuch as it greatly diminishes the dead weight and strain upon the iron beams; and the bricks themselves having been moulded to the proper curve, an even surface is obtained for the soffit or under side of the arch. This work then simply required pointing, the plastering of the arches being entirely dispensed with." The use of these bricks precludes the diminution of the depth of the ring of the arch from the springing towards the key or crown, as usually practised with the ordinary form of brick.

It is the common practice to set the ends of the beams resting on the side walls half an inch higher than that supported on the columns, and thus to compensate for the settlement of the masonry compared with the unyielding cast iron pillars.

214. The drawings, figs. 111, &c., represent the columns which have been employed in the warehouses lately erected at the new Marseilles Docks. The dimensions figured on the several drawings are in metres and millemetres, the values of which are given in English measures in a Table alongside; the details in this example are taken from a paper by Mr. Thomas Hawthorn, in the "Minutes of Proceedings of the Institution of Civil Engineers," vol. xxiv. It is stated by the author that the designs have been completed from a careful

study of the best and most recent examples in England, adapted to the commerce of Marseilles.

The pillar, it will be seen from the general elevation and section, consists of a central hollow circular shaft, with four vertical ribs or fins projecting externally from it, and these again stiffened at intervals by horizontal gussets, which fill in the four right angles between the vertical ribs; at each end the pillar has an octagonal disc, and both are united to the vertical ribs, which are curved outwards so as to give a fuller area for the bearings above and below; these octagonal discs are further strengthened by four bracket pieces, applied vertically at the middle point of the angles between the several ribs, and projecting also to the outer edge of the disc. The total sectional area of metal in the lowermost pillars is about 47 square inches; and in the two upper stories, which are of the same dimensions, 25 square inches. This quantity, if put into the form of a hollow cylindrical pillar, whose external diameter circumscribed the outer points of the vertical ribs at the middle of the height, would give a thickness of $1\frac{1}{8}$th inch for the lowermost, and $\frac{13}{16}$ths of an inch for the two upper stories; the dimensions of those supporting the intermediate floors lying between these extremes; the proportion of the diameters to the lengths ranging from one-eleventh to one-fifteenth. The columns are at a distance of 4.75 metres apart from each other throughout the whole building in both directions. One column simply rests on the top of the other, the two ends being turned in the lathe; while the girders rest on the lower flange of the upper column, being tied together by bolts passing through the column; what has been named hitherto the connecting piece is in this design cast in one with the upper pillar, instead of the lower. The columns were all cast vertically; they are 4 metres in height, and were tested to support a vertical load of 8 kilograms per millemetre square, which is equal to $5\frac{1}{4}$ tons per square inch. The total number of pillars used in the construction of these warehouses was 1520, and their weight 1616 tons.

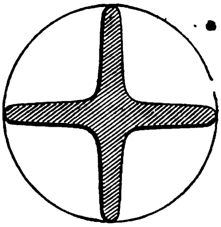
The girders are of double T-iron, rolled in one piece, 0.5 metre deep, and 4.53 metres long. The vaulting or segmental arches between the girders are built of hollow bricks, 6 in. deep, the spandril spaces being filled-in with cement, and made perfectly level on the upper surface, on which a layer of asphalte, half an inch in thickness, is laid for the floor surface.

The floors are constructed to carry 2 tons per square metre, or about 416 lbs. per square foot; but from the care taken in the selection of the materials and in the construction, they would probably carry much more with safety. The weight above named would give 45 tons on each floor for each pillar. The comparison of the column shown in this example with one of a cylindrical form involves many points of contrast, such as the greater risk of obtaining sound castings in so comparatively complicated a form, and having so many re-entrant angles, from which the simpler form of the cylinder would seem to be exempt; and, secondly, the relative expense of making the patterns and moulding may deserve mention. It also remains to be proved experimentally, on a large scale, that the distribution of the metal in this form gives a higher ultimate resistance than when the more usual cylindrical pillar is employed.

Mr. E. Hodgkinson's experiments, which have been given in Table § 148, No. 5, would seem to militate against any such forms of column; but no experiments on a large scale have been published on this identical description of pillar.

215. Such is the rapidity and economy with which goods can now be elevated and lowered by hydraulic hoists or steam machinery in warehouses, that no objection can be raised against the design of a structure such as this on the ground of being too high. But there is another point, and a very important one, viz., that when warehouses are carried up to a great height, so much more strength is required in all parts sustaining vertical load, that the economy of that plan of building them is doubtful, unless the ground space was extremely limited. The dimensions of this block of buildings were 1200 feet in length by 123 wide and 117.15 feet high, so that the horizontal area covered in is 147,600 square feet, and the cubical content 17,291,000 cubic feet. The total cost, including the hydraulic hoists, &c., amounted to £500,000, which gives £3.32 per square foot, and 6.94 pence per cubic foot. The thickness of the outer walls, which are of limestone, is at the vaults 4.103 feet; at the ground floor, 3.544 feet; and, gradually diminishing at each story, is 1.903 feet at the sixth story.

216. The annexed woodcut represents a cross section of a pillar, that may often be observed in our dock warehouses, when they have to support only a single floor. The circle shows the horizontal strengthening plate, which is inserted at two or three points in the height of the pillar, to stiffen the



vertical ribs and keep them well in place; the upper and lower flanges at the base and head of the column are sometimes of the same form, the necessary screw bolts passing through holes cast in the quadrantal spaces; or, if intended for carrying timber girders, they are given at the head a rectangular form, having the same breadth as the intended beam and a length sufficient to support its scarf or halving; vertical cheeks are also cast on each side, to give lateral security; the base also may be of a square form, or such as is best adapted to the particular support employed. The diameter of the pillar is generally made somewhat greater at the base, increasing gradually from the neck at the head, downwards. Doubtless the designers of this form of column are aware of the very small efficiency of the metal about the axis to resist fracture by flexure, compared with the same when in the form of a hollow cylinder; it may, however, be recommended by some facility in casting and moulding, as no core is required, and the greater readiness with which the surface may be examined for flaws and imperfections in casting; it may also be useful from the facility with which brackets may be attached by screw bolts at any part. At railway stations this section has been applied to the supports of the roof of carriage sheds, and it is convenient, as horizontal quartering of timber, to support the vertical sheeting boards at the sides, may be so firmly and easily affixed to pillars of this form.

217. The Erith Iron Works of Messrs. Eastons, Amos, and Anderson, are situated near the town of that name, on the south bank of the Thames, and stand upon an area of 13 acres; they communicate on one side with the North Kent Railway by a private branch, and with the river by a staging, alongside which the largest lighters can be moored; and it may be mentioned, as showing the extent to which facilities in manufacturing are carried at the present day, that this firm intend to communicate by railway with their old-established works, "The Grove," Southwark, a siding being laid out into that establishment; and a private telegraph, the wire being carried along the lines of railway, completes the intercommunication between the two establishments. The proprietors, being in possession of the ample area we have mentioned, adopted a design for the workshops which fulfils very well every requirement for manufacturing the largest and heaviest description of work.

However necessary it may be in densely populated districts to erect lofty buildings, many stories in height, with massive walls, so as to economize valuable ground space, in this locality it was obviously most consistent with the two objects of rapid and cheap construction and of future economy in working, to erect a series of pillar-supported sheds immediately contiguous and parallel to each other, but at the same time independent, and capable of being extended as future requirements would dictate, either by lengthening those already commenced, or by adding laterally to the number already constructed.

218. In Plate XI., fig. 116, is shown a transverse section of two of these sheds; and in figs. 117, 118, drawings representing in elevation and side view the arrangement of the pillar supports on a larger scale. The remaining figures show the details of the pillar in part sections, elevations, and plans, at one inch to a foot.

The span of the roofs of the several sheds is 42 feet, centres, which gives sufficient width either for the casting house of the foundry, the boiler shed, or for making bridge girders, and the placing and working of large tools.

The supporting columns are, it will be perceived, in two stories, the lower being a twin or duplicate column, joined at the base plate and at the head, so as to form one casting, and the upper a single column, the base of which rests on the centre of the cross head of the lower or twin column, to the flange of which it is firmly bolted.

The transverse distance of the several lines of pillars is, of course, the same as that of the span of the roof, namely, 42 feet, centres; and the longitudinal distance of the several columns in each range is 10 feet, centres, at which distance, consequently, the principals of the roof are placed; and it has been so designed as to avoid the need of having intermediate principals; small timber purlins, 2 inches by 3 inches deep and 12 inches apart, centres, supporting the roof covering.

219. The base plate of the twin column, fig. 119, is a rectangular plate, 3 feet 6 inches by 1 foot and 1 inch thick, cast in one with the pillars, which are externally octagonal in form, 8 inches from face to face, and having the internal diameter 7 inches; they are further united at the base by an intermediate vertical web ¾ inch thick, having an upper flange 4 inches wide and ¾ inch thick, which not only unites the pillars, but also forms the base into a strong beam 8 inches deep; an elongated bolt hole, 3 inches by 2 in. is cast in the centre, and the vertical web and upper flange enlarged to admit of this opening.

The base plate rests on a mass of brickwork, 4 feet by 2 feet, and about 2 feet 6 inches deep, which is built on a foundation of concrete of variable depth, the ground being, as might be expected from its alluvial character, very irregular in its powers of carrying weight. A single holding-down bolt, passing through the opening in the web, secures the base plate, its diameter being 1½ inch, and length about 3 feet 6 inches, fastened below to a cast iron "footlock" plate, built into the brick foundations, a washer and nut tightening it up above; the oval form of the bolt hole admits of a slight adjustment of the base when placing it on the upper bed of the brickwork. The base of the columns is placed 12 inches below the surface of the floors, in order to allow the rain water branches to be led into the drains without appearing above the floors, and without having recourse to the objectionable practice of bringing bends in through the brickwork underneath the columns. The rain water of the roof is passed down through one only of the twin columns, the other being blocked up with a piece of timber at the upper end of the internal hollow; the lateral opening in the lower end of the other, 5 inches in diameter, with its socket, is shown in the drawing. In the vertical section fig. 120, the hollow of the bottom of this column is shown filled in with a flanged disc secured by a rust joint, to obviate any injury from leakage into the foundation. At the respective heights, as figured in the drawing, connecting shelves are placed between the pair of columns, and in one casting with them; they are used to carry the countershafts and the long shafting. Their dimensions are fully marked on the detail drawings. The upper shelf is provided with lugs and bolt holes to secure the pedestals in which the shafting works, the section of the shaft being drawn in place.

The head of the twin pillars also forms a connecting shelf, having an upper flange, 3 feet 6 inches by 1 foot, and 1 inch thick; it is of a hollow, trough-like form, 9 inches in depth, acting as a beam to support the central pillar carrying the roof, and conveying the rain water from the central single pillar to the twin pillar left open for its discharge.

The upper column is also octagonal in its external form, 6 inches from one face to the opposite, and internally a hollow cylinder, 5 inches in diameter; its base plate has an area of 1 foot by 14 inches, and 1 inch thick, strengthened in its connexion to the pillar by gussets, and firmly held to the upper flange by four screw bolts, 1 inch in diameter; the pillar rests upon a bed of timber ½ inch thick, which obviates the expense of planing the surfaces of metal; and in the vertical section, fig. 126, it is shown that the column is carried down below the under surface of its flange for a depth of 1 inch, effectually obviating any leakage of the rain water, which is thus led past the upper surface of the head of the twin columns, and in superimposed columns this practice is now always adopted. Even with a rust joint, when a simple flange is used without the short pipe above described, injurious leakage is found to take place. The head of the upper pillar is formed by a flange, 1 foot 8 inches by 10 inches, strengthened by gussets, 7 inches by 9, and ¾ inch thick; raised edges, ¾ in. by ¾ in., are cast on each of the longer sides of the upper surface, intended to secure laterally the cast iron shoes to which the roof principals are attached; these are secured vertically by two hook-head bolts to each, ⅝ inch square; the flange and the raised edge are carried externally around the four openings left in the casting for the hook-headed screw bolts. The vertical section, fig. 129, shows the shoes and part of the principals in their relative position in side elevation, and also represents in section the method of conveying the rain water from the roof. A cast iron trough is placed with its two transverse upper edges on the level of the valleys, and the others overlapped by the roof covering; a circular opening 3¾ inches diameter is left in the V-shaped bottom, and a pipe of the same internal diameter, having flanges adapted to fit the shape of the outside of the bottom of the well, to which it is fastened by ½ inch screw bolts; this pipe enters the internal hollow of the upper column for a depth of 1½ inch; the trough rests upon and is fastened to the principals on each side.

The roof is compound in structure: a rafter, 3 inches wide by 6 deep, has on each side a plate of wrought iron, 4 inches deep by ⅜ inches; the inclined tie bars, one on each side of the rafter, are 2 inches by ½ inch, having circular ends, 4 inches in diameter with eyes adapted for a pin, 1¼ inch diameter, which passes through all, secured with a head on one end, and a cotter, ¼ inch by ¾ inch on the other. The tie bar takes the outward horizontal thrust of the principal, so that a vertical pressure alone acts upon the shoe.

Angle brackets are cast upon the longitudinal faces of the upper pillars, of the dimensions shown in figs. 126, 128, and their respective end views, 126 *a* and 128 *a*. Quarterings of timber are bolted to the brackets, and stretch horizontally from one pillar to the other; and corrugated iron sheeting, being screwed to the timber in the vertical plane, forms a complete screen between each shed.

220. Returning again to the level of the flange of the head of the twin columns, we have on each end the bearings of the longitudinal cast iron beams, which are shown in elevation in fig. 117, at A, A, and at 1 inch to a foot in fig. 130, and in transverse section in fig. 125.

The total length of each beam is 10 feet, and the clear span 9 feet, the depth at the centre being 1 foot 3 inches, and at the end 1 foot 2 inches, the upper flange being perfectly level; the increase of depth is due to an increase in the thickness of the lower flange. As in the case of the upper pillars, and with the same object, the beams rest on a $\frac{1}{2}$ inch bed of timber; the end of each beam is secured to the upper flange of the pillar head by 1 inch screw bolts, oval holes being left in the pillar head and in the lower flange of the beam, $1\frac{3}{4}$ by 1 inch, those in the beam being at right angles with the holes in the pillar head. The upper and lower flanges project beyond the vertical web, in order to allow a close fit to be worked on them when in place, as shown in fig. 130; the web is stiffened by gussets between the flanges, one on each side at both ends, and bolt holes are left on the inner gussets, by which the girders are secured longitudinally, so that they form a continuous tie from end to end of each range of columns. Their proper duty is, however, to form a rail, on which a 20-ton travelling crane works, and for this purpose the upper and lower flanges are formed as shown in the transverse section. The chief advantage of the system of twin columns is, that they can thus be made to receive the weight from the wheels of the crane fairly in the direction of the centre line of each pillar. This has commonly been done in workshops by casting brackets on massive circular columns placed in a single line; but such an arrangement throws a cross strain on the column, and is wanting in security, partly on that account, and partly also from the bracket breaking off. The octagonal form is extremely convenient for attaching miscellaneous small brackets, pedestals, &c., that are invariably required where machinery is used and where there are no walls or roof timbers to carry counter shafts.

221. So completely do the twin pillars supply the needful lateral stiffness, that they have been used as a staging to support the travelling crane outside the roofed-in part of the boiler shed down to the water's edge; 8 cast iron cylinders,

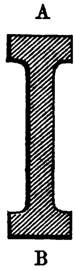
3′ 6″ in diameter, being sunk down to solid ground in the waterway in continuation of the lines of the pillars; wrought iron girders, of span sufficient to admit of a lighter, passing underneath, between the cylinders and the wharf. By this arrangement a massive piece of finished work may be taken up at the furthest end of the shed, and let down into a lighter, so that it is afloat on the river in a few minutes.

Towards the river the ground gets very soft; and, instead of isolated blocks of brickwork, a trench, 6 feet wide, and 4 feet deep, has been filled with concrete, in a continuous mass, on which the brick blocks under the twin columns are set.

The only alteration in the castings of the twin columns was in those of the four corner columns of the tower in the boiler shed, and these are held down by two bolts, each passing through snugs outside the pillars; wider and deeper foundations are also provided for them.

The advantages of twin columns in workshops have been secured, when not in one casting, by bolting them to a common base plate and common upper piece, proper flanges, &c., being provided on the columns; and this will be necessary when the height much exceeds that of the pillars in this example.

222. The woodcut on the left hand represents the horizontal section of the form of cast iron pillar adopted in supporting the roofs of the building slips in the Royal Dock yards. The longest line in the engraving—that is, in the line A B—is in the transverse plane, at right angles to the length of the building slip, and therefore in the direction of the outward thrust of the roof. This form of pillar, though ill adapted to resist flexure in the direction of its breadth, is suitable to its peculiar duty, having only to sustain the weight of a light roof, the unbalanced outward thrust of which (if any) it must receive in the direction A B, in which it has the greatest power of resistance. This form is also useful as giving facility in attaching brackets or shelf pieces for gangways, frames for boarding, or sashes, &c. The diagram, transverse section, fig. 131, shows the overhanging portion of the roof, the weight of which, resolved in the horizontal, tends to balance that of the main span. A pillar of very similar section has been employed at the workshops erected parallel to the graving docks in the port of Dublin.



223. Under circumstances in which, from their intended duty, cast iron pillars may not have any very great vertical load to sustain, and yet require to be of great height and great

diameter, to resist the contingency of a lateral force applied at the head, we find that in such applications the pillar is commonly cast in several lengths, with internal flanges and bolt holes.

The Exhibition Building in London, for 1862, contained remarkable examples of such pillars of cast iron, in the vertical supports of the inner line of the ribs of the great domes. These columns are 100 feet high, and 2 feet diameter, perfectly uniform and cylindrical, cast in four lengths of 25 feet each, with interior flanges and bolt holes, the connecting surfaces being truly faced—fig. 132. These several lengths were successively elevated into position; and a man was let down the interior of the column with a light, who screwed up the bolts of the successive joints. These columns were clustered together at different points in the circle, and joined laterally by distance pieces and bolts, so that each assemblage acted together, and gave mutual support in resisting incipient flexure.

If we suppose the metal to have been 2 inches in thickness, we shall then have the ultimate strength of each of these pillars, supposing it to have acted as a single casting,

$$42.35 \times \frac{24^{3.5} - 20^{3.5}}{108^{1.63}} = 42.35 \frac{67723 - 35777}{2062.9} = 656 \text{ tons};$$

this being on the supposition that the joints and flanges are as strong as the solid metal would have been if it were practicable to have cast such a column in one length. But under an extreme vertical load we should most probably have the incipient deflection concentrated about the joints, and some of the bolt holes on the side in tension give way before the calculated load had been brought on the pillar.

"The pillars supporting one side of the great roof at the Joint Station at Birmingham are 35 feet in height, and 2 feet in diameter, with a thickness of one inch; they are cast in two pieces, with a base plate in addition; these pieces are connected together by screw bolts through internal flanges on the ends of the pieces, all of which were accurately faced in a lathe, and slightly grooved. The columns weigh about 5¼ tons each, and serve as pipes to convey away the rain water; they are fixed on solid concrete foundations, but are not bolted down. Upon the planed surface of a bearing plate, fitting on the top of each column, is placed a frame of wrought iron rollers, upon which the foot of the truss rests, the rollers being intended to admit of the contraction and expansion of the metal in the truss, caused by the variations of temperature." ("Minutes of Proceedings of Institute of Civil Engineers," vol. xiv., p. 253, &c., by Joseph Phillips.)

224. The application of wrought iron beams in fire-proof floors in lieu of cast iron has been well exemplified in the fine building recently erected by Mr. Fairbairn, in Dublin, as a sugar refinery, for Messrs. Bewley, Moss, and Co. "This building is probably one of the most important yet constructed with brick arches in wrought iron beams; and it may well be referred to as an example of what may be done by the introduction of a material free from flaws, and much lighter than cast iron. The weight which these floors and beams have to sustain, when loaded with moist sugar, has been calculated at 400 lbs. on the square foot, and the breaking weight of the beams is computed at 106 tons, uniformly distributed.

"It will be observed that the building is about 150 feet long, and 58 feet wide (figs. 133, 134, 135), with only one line of columns down the centre of the main building, and three lines in the lower floor of the cistern house. Through the head of each column (figs. 106, 107, &c., Plate X.) a tie rod, 1 inch square in section, extends from one gable to the other, and is built into the arch. Transversely with the tie rods are four bolts, which secure the ends of the beam to the columns. To the top flanges of the beams are riveted 6 flat tie rods, 2½ inches by ⅜ of an inch, one built into the walls at each side, and the others equally divided between them and the centre of the building; these are to retain the beams straight in their places until the arches are turned from one end of the building to the other. From this it will be observed, that the tie rods unite the arched floors and the walls on each side, forming an excellent bond, giving tenacity and unity to the structure *en masse*."

"During the time of building or turning the arches, it is, however, necessary to be careful that, before the centres are struck, the thrust of the arch does not force the beams out of line by lateral pressure, as any undue force in that direction would prove highly injurious, and seriously impair the strength of the beams. It is necessary to be very cautious in this respect, as wrought iron beams, being more flexible than cast iron, are liable to yield by lateral pressure." ("Application of Iron," &c., 3rd edition, p. 166.)

The pillars in the main building have the several thicknesses figured, decreasing from 1½ inch on the ground floor to

$\frac{5}{8}$ of an inch on the two upper lines. The roof covering has been formed by a cast iron water tank; the heights of the several stories and columns are given in figs. 133, 135.

225. Many other interesting applications of cast iron in direct compression might be added; that important branch of the subject which includes the application of cast iron in lieu of piers of masonry in wrought iron girder bridges, and of timber piles in jetties and landing stages, will be explained and illustrated in the Examples, in Part II. The Tables of the values of $d^{3.5}$ and $l^{1.63}$ will be given at the end of the volume.

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226. **Malleable Iron.**—*The ultimate resistance of Wrought Iron to a force of compression is equal to from 16 to 18 tons per square inch, and is proportional to the area of transverse section in action.*

Experimental Proofs.

227. The experimental proofs are derived from the results published by Mr. Hodgkinson in 1840, and from his later researches in 1845, which, although directed to the construction of beams and bridges of wrought iron, have been given in the present Part.

The peculiar characters of this material do not admit of the same precision in judging from external appearances, when crushing commences, as in the case of cast iron, in which there is an abrupt change in the metal by the complete disunion of the parts: a prism of the best soft wrought iron, on the contrary, begins to be very slightly flattened and enlarged in diameter with about 12 tons per square inch, but we never observe the wedge-shaped fragments which break off suddenly from cast iron when its ultimate resistance has been overcome, as described in §§ 111 to 117. Nor do the peculiar applications of wrought iron in practical construction give any high value to experiments on the crushing of short solid prisms or pillars of this material. It is chiefly used when in compression, not for columns, to which cast iron is almost exclusively applied, but in the flanges of beams, in the form of rectangular cells, or circular or oval tubes and flanges constructed of rolled plates, united by riveting. The experiments, given below, which we do possess on short prisms, we owe to Mr. E. Hodgkinson, to which he was led in his endeavour to clear up some of the difficulties he had met with in the results of his experiments on cast iron pillars, as mentioned in pages 122, 123, &c., and not with a view to establishing rules for practical construction; for it was not until some years subsequently that the great development of the use of wrought iron in construction, consequent on the extension of the railway system, took place, and which rendered necessary a more extended inquiry into the properties of this material.

228. Three small cylinders of the best Staffordshire wrought iron were carefully prepared. They were exactly 2½ inches long, and 0.62 inches in diameter, their ends being turned flat and perpendicular to the axis; having been subjected to compression with their ends well bedded, they were, after bearing certain considerable weights, tried by a gauge which had exactly fitted them before the commencement of the experiment. The results are given in the following Table.

229. TABLE, *showing the Effect of great Compression upon short Cylinders of Wrought Iron, and the Ultimate Resistance per square inch. Length* 2½ *inches, diameter* 0.62, *ends flat.*

Number of Experiments.	Weights, laid on, in lbs.	Weight per Square Inch.	Reduction in Length.	Remarks.
		lbs. tons.	Inch.	
1	6222	20610 = 9.2	0.0?	No alteration; or, if any, extremely small.
1	7342	24319 = 10.9	0.015	
1	8462	28028 = 12.5	0.02	Diameter sensibly increased, cylinder not bent.
2	12444	41218 = 18.4	0.04	Diameter sensibly increased, from 0.62 to 0.63 inch, slightly bent.
3	18667	61830 = 27.6	0.16	Bent a little, diameter increased to 0.65 inch.

230. We see from the above Table that 9.2 tons per square inch on the cylinder, 0.62 inch in diameter, produced no sensible change in it; but that 10.9 and 12½ tons per square inch, particularly the latter, produced very obvious changes. The experiments bring out very clearly the wide difference in the character of cast and wrought iron under great compression. Instead of that sudden splintering up into fragments of a wedge-like form, without any previous warning or any per-

[illegible] change of external form, as was exhibited in the short [illegible] of cast iron, we find wrought iron, from its great tenacity and ductility, showing signs of a decided alteration in the dimension of the cylinder, which would continue to sink down to any degree under the increased application of weight. The further study of the compression of wrought iron, as applied to beams, in Part II., will also show that this material can be used in forms, such as hollow cylinders and rectangular hollow cells or tubes, which enable it, although never resisting beyond the values assigned above, to oppose compression far more efficiently than as a solid pillar.

The following experiments will enable us to determine the laws of the strength of solid wrought iron pillars as depending on their dimensions Although the range of the relations of length to diameter may not be as wide as in those of cast iron already given, yet the analogies with those derived from the results on cast iron may, with the Table, in some measure meet the wants of practice.

231. Table, *showing the Ultimate Strength of Solid Uniform Cylinders of Wrought Iron to a Force of Compression.*

		Both ends rounded.		One end round, one flat.		Both ends flat.	
	Length.	Diameter.	Breaking Weight.	Diameter.	Breaking Weight.	Diameter.	Breaking Weight.
	Inches.	Inches.	Lbs.	Inches.	Lbs.	Inches.	Lbs.
I.	90¾	1.017	1808	1.02	3355	1.02	5280
II.	60½	1.015	3938	1.03	8137	1.02	12990
III.	30¼	1.015	15480	1.015	21355	1.015	23371
IV.	30¼	1.015	15480	1.015	21187*	1.015	25387*
V.	15⅛	1.005	23535	1.005	26227	1.005	27099
VI.	30¼	0.52	1260				

232. The pillars in each horizontal line in the above Table are of equal length, and nearly of equal diameter, the only essential difference being in the form of their ends. Taking the longer pillars in the two upper lines, the strengths of the different forms are nearly in the ratio one, two, three; the middle one, which has one end flat, and the other rounded, being an arithmetical mean between the other two.

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* These had discs on the flat ends.

Thus the pillars 90¾ inches long broke with the weights

	1808	3355	5280
which are as	1	1.8556	2.92

In the shorter columns this ratio does not obtain; but even with them the pillars having one end rounded, and the other flat, are, as to their ultimate strength, arithmetical means between those of the other two forms, as in this Table.

	Mean from Strength of Pillars, both ends rounded, and both flat.	Strength of Pillars, one end round, and the other flat.
I.	3544	3355
II.	8464	8137
III.	19426	21355
IV.	20434	21187
V.	25317	26227

In the shorter pillars, in the last three horizontal lines, this ratio of one, two, three, does not obtain, they having been slightly crushed by the weight; and it would appear that the difference of the strengths of the three forms of pillars becomes less, according as the number of times which the length of the pillar exceeds the diameter decreases. But, whatever may be the ratio of the strengths of pillars with rounded ends to those with flat ones, the strength of those with one end rounded and one flat is nearly an arithmetical mean between them.

233. To determine the inverse power of the length to which the strength is proportional, we may proceed in the same manner as in § 126, p. 89, where the power for cast iron pillars was deduced.

Let *n* be the sought power to which these lengths must be raised to make them in the inverse ratio of the breaking weights, comparing the lines I. and IV., ends rounded.

$$\begin{array}{l} \\ \text{or} \end{array} \quad \left.\begin{array}{r} 90{\cdot}75^n : 30{\cdot}25^n \\ 3^n : \quad 1^n \end{array}\right\} 15480 : 1808$$

or
$$3^n = \frac{15480}{1808} = 8.562$$

so that
$$n \times \log 3 = \log 8.562$$

or
$$n = \frac{\log 8.652}{\log 3} = \frac{0.93258}{0.47712} = 1.9546$$

consequently, from this single experiment,

$$90.75^{1.9546} : 30.25^{1.9546} :: 15480 : 1808$$

The following Table gives the average result of the value of n from pillars sufficiently long not to have been crushed with the breaking weight.

TABLE, *showing the Inverse Power of the Length in Wrought Iron Pillars.*

Lengths of which the Strengths are compared.		Description of Pillar.	Inverse Power of the Length.	
Inches.				
90.75	30.25	Rounded ends,	1.9546	Mean 2·0316
90.75	60.50	Rounded ends,	1.9199	
90.75	60.50	Flat ends,	2.2203	

whence it appears that the strength is in wrought iron as the inverse square of the length nearly, differing in this respect from cast iron.

234. In like manner, as regards the powers of the diameters, to which the breaking weights are directly proportional, comparing pillars of the same length, and with ends rounded, in lines VI. and III., § 231, we find that a pillar 0.52 inches in diameter had, from a mean of three experiments, its greatest resistance overcome by 1260 lbs.; and a pillar of the same length, and 1.015 inches in diameter, also from a mean of three experiments, sunk down under 15480 lbs. The ratio of the diameters will be more conveniently expressed as

$$0.52 : 1.015 :: 1 : 1.9519$$

so that, if we put m for the index, we have the direct ratio

$$1^m : 1.9519^m :: 1260 : 15480$$

Hence $$m \times \log 1.9519 = \log \frac{15480}{1260} = 1.08940$$

and $$m = \frac{1.08940}{0.29047} = 3{\cdot}75073$$

consequently, from this single experiment,

$$0.52^{3{\cdot}75} : 1.015^{3{\cdot}75} :: 1260 : 15480$$

This index is very nearly the same as the mean value of that deduced for cast iron pillars with rounded ends, § 131, pp. 107, 108; and § 133, p. 112.

235. In the absence of more extended experiments on wrought iron pillars with flat ends, we may, therefore, with Mr. Hodgkinson, assume, from the almost complete identity of the index of the diameters when the ends are rounded, that by analogy, they would also be the same if the ends were flat. Thus, in solid wrought iron pillars, putting *W* for the breaking weight—that is, the weight which overcame the greatest resistance of the pillars, and under which they sank down—we have

$$W \text{ proportional to } \frac{d^{3{\cdot}76}}{l^2}.$$

when the ends are rounded; and

$$W \text{ proportional to } \frac{d^{3{\cdot}5}}{l^2}.$$

when both ends are flat. Consequently, in each of these cases, as in all other proportional quantities, there is some constant number, by which if the lesser be multiplied we shall have a result equal to the greater. To illustrate this, if in the case of the several pairs of numbers, 19 and 3.8, 36 and 7·2, 48 and 9·6, which are proportional, we multiply the lesser ones by 5, we shall have them, respectively, equal to the greater. This coefficient is found for all of them by dividing out any pair of the numbers by the lesser, and the ratio will stand 5 to 1. So also the numbers which represent

$$W, \text{ and the quantity } \frac{d^{3.5}}{l^2}.$$

being in every case in the same ratio, we can obtain a constant multiplier; and therefore

$$\text{Breaking weight} = \text{Constant} \times \frac{d^{3.5}}{l^2}$$

[illegible] what has been said above, will evidently [illegible] weight of the unit column; for when d and l [illegible] unity, their values, raised to the powers 3.5 [illegible] respectively, will still remain the same number. The [illegible] use of this so found constant will be, that we can [illegible] any one of the three quantities W the breaking [illegible] the diameter, and l the length, when the other two are given. It is not necessary that d and l should have the same unit; it is usual in practice to express the diameters in inches, and the lengths in feet, so that the unit column is that which has a diameter of one inch, and a length of one foot.

236. The first step in the computation of this constant is to calculate from some experiment in which we have a pillar of a known diameter, as, suppose, 0.52 inches, and breaking weight W, that of another, which call W_1, one inch in diameter, each having the same length.

The second step will be from the ultimate resistance W_1, of this pillar, one inch in diameter, to calculate that of another, having also a diameter of one inch, and a length of one foot, by an inverse proportion; and this so computed breaking weight will be the constant sought.

Thus, with a round-ended pillar, 1.017 inches in diameter, and 90¾ inches long, that is, 7.5625 feet,

$$1.017^{3.76} : 1^{3.76} :: \left\{ \begin{array}{l} W : W_1, \\ \text{or } 1808 : 1634 \end{array} \right.$$

and again the strength of the pillar one inch in diameter, and 7.5625 feet long, will be to that of a pillar of the same diameter, and one foot long, in the inverse ratio of the squares of the lengths in feet:

$$\frac{1}{7.5625^2} : \frac{1}{1^2} : 1634 : x$$

or, $$1 : 57.886 :: 1634 : x = 94586$$

From an average of pillars 90¾ and 60½ inches long, with rounded ends, Mr. Hodgkinson deduces by the above method $x = 95848$ lbs., equal to 42.8 tons; and from those of the same lengths as above, and with flat ends, the average was $x = 299617$ lbs. $= 133.75$ tons—a result from this one pair of experiments three times greater than that deduced for cast iron pillars with flat ends.

Mr. Hodgkinson states that, in order to compare the strengths of pillars of the same size in these two different materials, he had computed the strength of a pillar of the same diameter and length in each body, taking results from the longest pillars in order that they may not be partially crushed by the breaking weight, and found that, calling the strength of cast iron 1000, we have for the relative strength of wrought iron 1745.

It is by means of this latter ratio that the requisite dimensions of solid wrought iron pillars are determined, when it is found advantageous to make use of this material in preference to cast iron. The average of the greatest resistances of the cast and wrought iron pillars in the first four lines of the Table, § 239, page 234, is for cast iron 1.873 tons, and for wrought iron 3.259 tons, which are in the ratio above given.

237. The following statement is a summary of the several propositions which have now been proved experimentally as to the greatest resistance to compression of short specimens of cast and wrought iron, and also of their ultimate resistance to tension.

	Tension.	Compression.
Cast Iron,	about 7 tons per square inch.	About 42 tons.
Wrought Iron,	20 to 25 ,,	About 16 to 18 tons.

The ratio in the upper line being about one to six, and in the lower nearly that of equality.

We also see that, although wrought iron is nearly three times stronger than cast iron under direct tension (and with this application of force, the ratio of the length to the diameter of the specimen is immaterial), yet that under direct compression the relation between them is reversed, cast iron being nearly three times the stronger of the two metals, when short specimens are used.

238. As the length of the experimental pillars is gradually increased, we find, however, that the resisting power of the wrought iron comes up to, and eventually overtakes that of the cast iron.

The following Table will enable us to compare at a glance the relative ultimate power of resistance of each material at different ratios of diameter to height. The diameters of each of the specimens was so nearly one inch, that it has in all of them been assumed to be that exact quantity. Each result stated has been taken from some of the Tables given elsewhere in this work.

239. TABLE, *showing the relative Ultimate Strengths or Greatest Resistance, of Cast and Wrought Iron Solid Cylindrical Pillars, all one inch in diameter, ends* flat

Mode of failure.		Ultimate Resistance of		Ratios.		Strength per sq. inch.	
	Lengths.	Cast Iron.	Wrought Iron.	Cast Iron.	Wrought Iron.	Cast Iron.	Wrought Iron.
By Flexure only.	Inches.	Tons.	Tons.				
	120	0.994	1.489	100	150	1.3	1.9
	90	1.644	2.357	100	140	2.1	2.9
	75	2.074	3.394	100	160	2.6	4.3
	60½	2.781	5.799	100	208	3.6	7.1
	30¼	9.067	10.433	100	115	11.5	12.9
				100	100		
Partly by Flexure and by crushing.	20.17	14.198	9.90	100	70	18.1	12.7
	15⅛	17.969	12.07	100	67	22.9	15.4
	7½	20.107	12.60	100	62	25.7	16.0
By crushing.	2½	33.00	14.14	100	43	42.0	18.0
	1½	33.00	14.14	100	43	42.0	18.0

240. The column of lengths in the above Table gives also the ratio of the length to the diameter, since the latter was in each case one inch. The columns of the Ratios of the strength in the two metals plainly indicate that at about 30 to 1—namely, the height at which commences the yielding of pillars by flexure only—both pillars have equal resisting powers; above this point the ratio is in favour of the wrought iron, and below it rapidly reversed.

241. The point in the Table at which the change in favour of wrought iron takes place helps us to an explanation; in fact, the high tensile resistance of this material, compared with cast iron, is then brought into effectual action in resisting flexure. Now, in solid circular pillars of wrought iron the line dividing the part of the transverse area in tension from that in compression cannot be far removed from the diameter, on account of its nearly equal resistance to these two forces, while in cast iron, from its great inequality in resisting each of these same forces, the line of separation must be nearly

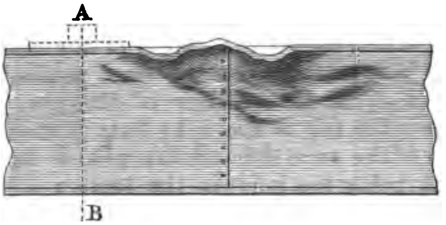
at about the distance of half the radius from the diameter—*vide* § 138, p. 120—and therefore the area of the part under tension considerably greater than in the wrought iron pillar of the same diameter, yet this cannot compensate for its comparative weakness, having, as we have seen, only one-third of the tensile resistance of wrought iron; and, again, the distance of the centres of tension and compression in a wrought iron pillar will (within the limits of the Table) be greater than the interval between the same points in a cast iron pillar; so that, in both points of view, a long flexible wrought iron pillar (up to about 15 feet) is stronger than one of cast iron of the same dimensions; what is named the centre of tension being the point in the tension area in which if all the resistances were accumulated, the result would be unaltered; and so of the centre of compression; the distance between these two points will evidently be the leverage by which the pillars resist the action of the load causing flexure.

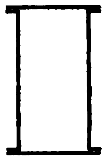
The price of wrought iron is higher relatively to that of cast iron, than the numbers above given representing their respective strengths, so that no advantage in this respect can be claimed for the former. Other considerations than mere economy will determine the adoption of it in preference to cast iron in constructing pillars.

242. The subject of the laws of the greatest resistance of wrought iron to a direct force of compression received no further attention for several years, until about the year 1845, when the necessity of determining in detail the dimensions of the tubular girders of this material, intended to span the Conway River and the Menai Straits, became urgent. Model tubes of various forms in transverse section, as circular, elliptical, and rectangular, and of different spans from 17 to 34 feet, had been constructed of wrought iron plates riveted together, and loaded at the centre until they had been broken; but the results were by no means satisfactory or assuring; for that part of the beam which was under compression—namely, the top—yielded far under its known power of resistance by a new and peculiar mode of failure, variously described as undulation, buckling, crumpling, wrinkling, &c. In the words of Mr. E. Clark—"No one knew *à priori* the resistance of plates of wrought iron to buckling, which was a new fact altogether, and one not involved in any of the formulæ for beams hitherto employed. It, indeed, annihilated at once their practical utility; and, prominent as it became in subsequent experiments, it for a time threatened even to frus-

trate the consummation of the designs for crossing the great openings already mentioned on the line to Holyhead."

243. The wood engraving, which is an elevation of a portion of one of the rectangular tubes, at a scale of half an inch to a foot, with the transverse section on the right-hand side, represents the corrugation or undulation that took place in the top in one of the earlier experiments, and by which it failed. Though the undulations were not identical in every case, this drawing gives a good idea of what took place in other tubes. Some of them, indeed, failed by tension, tearing asunder from the bottom at a line of rivets; but this was easily met by an improved method of uniting the several plates, which were joined by a cover plate and double line of rivets, instead of a mere overlap joint, having a single line of rivets, as at the first. The vertical line joining the points A and B indicates the centre of the span, and the dotted lines show the bolts, with nut and washer, by means of which the successive weights were applied to the tube. The clear span of this beam was 17 feet 6 inches, the depth 18¼ inches, and the breadth 9½, with a thickness of about 0.21 inch, giving an area of 1.94 square inches in the top plate, which was the part that yielded to the compressive force in the manner represented. The rectangular form of the transverse section of this tube is shown in the second engraving, which is a vertical section through the centre. The final deflection of the beam at the centre was 0.93 inch, with the last applied weight of 3.04 tons. From simple computations, which we will not here anticipate, as they properly belong to Part II. of this work, it would appear that the compression per square inch which caused this undulation in the top plate was not more than the fourth part of the ultimate resistance of that area to a crushing force.





244. This crumpling up of wrought iron plates is but a modification of the phenomena of failure by compression, and

one that can only occur with a ductile and tenacious material, which does not separate into fragments, or suddenly break across, as cast iron is observed to do. From a careful examination of the engraving, it would seem that the crumpled part of the top plate may be considered to have acted as a pillar perfectly and rigidly confined at each end; but which, through some incidental comparative weakness either of the plate itself, or in the riveting by which it is attached to the sides, or from both causes coinciding together at the same place, as may possibly occur, is permitted to act as a column of the same material would do if unsupported laterally throughout its height. The length of the corrugated part would seem to be about 18 inches; and, as the thickness is under ¼ inch, the ratio of the length to the thickness is nearly 80 to 1; in the buckled portion of the top it must, therefore, be extremely flexible, and yield at a pressure per square inch small when compared with the ultimate resistance to crushing. The plates in lines IV. and VII., Table § 246, have a ratio of length to thickness of about 80 to 1, and they yielded by flexure with about 4½ tons per square inch. The remaining part of the top plate, if it continued without distortion, and was held truly in the line of the compressive force by its own strength and by its connexion with the other portions of the tube, would of course be capable of resisting crushing as a short specimen, and up to the full power of about 16 or 18 tons per square inch.

245. We can now perceive how well grounded was the anxiety which this unexpected and perplexing effect of compression on wrought iron caused to the engineer responsible for the proposed structures at Conway and the Menai Straits. It was at this conjuncture that Mr. Eaton Hodgkinson, so justly esteemed as an experimenter, was called in; and he, rightly judging that further information was required as to the resistance to compression of plates of wrought iron thicker than those which had been employed in the first experiments on the model tubes, proceeded to institute a series of experiments on the resistance to a crushing force of plates of wrought iron of different length, breadth, and thickness.

The laws of solid pillars as already given, though an important link in the complete investigation of the subject, are evidently not directly applicable to the circumstances under which the top of a tubular beam is destroyed by compression. It yet remained to investigate the effect of that strain on plates of various dimensions or thickness; and, lastly, the subject will become more intricate when the resistance of these plates to crushing, as combined in the forms of cells, whether circular or square, is considered.

This much has, however, been already settled, § 229, namely, that wrought iron is literally crushed by a force of from 16 to 18 tons per square inch of area ; and under no circumstances can any pillar (whose length is greater than its diameter) carry a weight exceeding this, which is the major limit of resistance to compression. We shall find that we can nearly approach this limit in long plates and cells, when suitable proportions have been adopted, and that it is in fact the test of well-applied material in any experimental inquiries on wrought iron beams.

The following Table gives an abstract of the extended series of experiments, 40 in number, undertaken with the view of determining the law of the strength of plates of wrought iron, as depending on their dimensions.

246. TABLE, *showing the greatest Resistance of Plates or Bars of Wrought Iron to a Force of Compression; ends turned* flat, *and perfectly bedded against the parallel surfaces between which they were compressed.*

	Length.		Dimensions.		Area.	Greatest Resistance.	Pressure per sq. inch.
	Ft.	In.	Ins.	Ins.	Sqr. Ins.	Tons.	Tons.
I.	10	0	2.98 ×	.497	= 1.481	0.545	0.364
II.	”	”	3.01 ×	.766	= 2.306	3.48	1.508
III.	”	”	2.99 ×	.995	= 2.975	5.68	1.911
IV.	”	”	3.00 ×	1.51	= 4.53	20.56	4.538
V.	7	6	1.024 ×	1.025	= 1.0496	4.56	4.354
VI.	”	”	2.983 ×	.5023	= 1.4983	1.61	1.076
VII.	”	”	3.005 ×	.9955	= 2.9915	13.22	4.425
VIII.	”	”	3.00 ×	1.53	= 4.59	40.96	8.923
IX.	”	”	5.86 ×	.995	= 5.8307	24.16	4.143
X.	5	0	1.024 ×	1.024	= 1.0486	8.08	7.709
XI.	”	”	2.98 ×	.507	= 1.511	3.78	2.502
XII.	”	”	3.01 ×	.767	= 2.309	13.37	5.79
XIII.	”	”	3.01 ×	.995	= 2.995	24.16	8.066
XIV.	”	”	5.84 ×	.996	= 5.8166	45.96	7.901
XV.	2	6	1.0235 ×	1.0235	= 1.0475	11.84	11.307
XVI.	”	”	2.9867 ×	.5026	= 1.5011	11.27	7.524
XVII.	”	”	3.01 ×	.763	= 2.297	28.48	12.396
XVIII.	”	”	3.00 ×	.996	= 2.988	39.56	13.239
XIX.	1	3	1.023 ×	1.023	= 1.0465	16.14	15.426
XX.	0	7½	1.023 ×	1.023	= 1.0465	22.74	21.733
XXI.	0	3¾	1.023 ×	1.023	= 1.0465	24.60	23.549

247. The design of this series of experiments is very clear. As to the lengths of the plates from lines I. to XXI. they are as the numbers, 4, 3, 2, 1, $\frac{1}{2}$, $\frac{1}{4}$, $\frac{1}{8}$. And as to the thickness, in the first four lines, which are of the same length, namely 10 feet, the width also being constant and in round numbers 3 inches, we have the third dimension nearly as the numbers, 1, $1\frac{1}{2}$, 2, 3. We can thus, from dimensions which are multiples and submultiples of each other, very readily ascertain the increase of strength due to the increase of thickness in long flexible plates under compression, the thickest of which, line IV., was to its length as 80 to 1.

So, also, with plates 7 ft. 6 in. long, if we select for comparison lines VI. and VIII., which have the same breadth of 3 inches, and the thicknesses, respectively, $\frac{1}{2}$ inch and $1\frac{1}{2}$ inch, we can obtain the index of the thickness to which the strength is proportional; that is, the law of the increase of strength.

248. Thus in lines VI. and VIII., if we put *m* for the unknown power of the thickness,

We have $(\frac{1}{2})^m : (1\frac{1}{2})^m$

Tons. Tons.

or $1^m : 3^m :: 1.61 : 40.96$

therefore, $3^m = \frac{40.96}{1.61} = 25.441$

whence $m \log 3 = \log 25.441$

and $m = \frac{\log 25.441}{\log 3} = \frac{1.40553}{0.47712} = 2.945$

In like manner, by other comparisons, as from lines XI. and XIII., we obtain an average very nearly equal to 3; the longer plates giving a rather higher value for *m* than the shorter, as we should have expected from what has been already stated in pages 120, 121 of cast iron pillars. Hence the strength of plates of wrought iron is proportional to the cube of the thickness; and thus a plate twice as thick as another of the same length and breadth is eight times as strong to resist failure by flexure.

249. The dependence of the strength upon the length of the plates may also be deduced from the Table. Thus, com-

paring lines XIII. and III., in which the breadth and thickness are very nearly the same—that is, 3 inches by 1 inch, and the length 5 feet and 10 feet, or one to two—we have the inverse proportion,

$$1^n : 2^n :: \overset{\text{Tons.}}{5.68} : \overset{\text{Tons.}}{24.16}$$

therefore $$2^n = \frac{24.16}{5.68} = 4.2535$$

whence $$n \log 2 = \log 4.2535$$

and $$n = \frac{\log 4.2535}{\log 2} = \frac{0.62875}{0.30103} = 2.08$$

250. An average from a number of comparisons worked out as above will give very nearly 1.9, whence we may assume the index n to be equal to the second power ; and thus, if the breadth of plates be the same, and the length l and thickness t vary, we shall have W the weight of greatest resistance.

$$W \propto \frac{t^3}{l^2}$$

If we compare lines V. and IX., and again X. and XIV., in which the thickness is nearly equal, we find that the breaking weights are proportional to the breadths, proper corrections being made on account of the small difference in the respective thicknesses, which are 1.025 and 0.995 inches Thus, finally, we deduce that plates used as pillars, and o. such dimensions that each fails by flexure, have their strengths represented by the expression,

$$W \propto \frac{bt^3}{l^2}$$

So that doubling the breadth of a plate simply doubles the strength, and doubling the length reduces the strength four times; but doubling the thickness, as was said above, increases the strength eight times; and if the thickness be increased only by the fourth part, the strength to resist destruction by flexure is very nearly doubled; for let x be the additional thickness, then $1^3 : (1+x)^3 :: 1 : 2$ and therefore

$$1 + x = \sqrt[3]{2} = 1.26 \text{ and } x = 0.26.$$

251. It also results from the same experiments that similar plates or pillars of wrought iron have their strengths proportional to the square of their lineal dimensions, the condition being observed that each is of such length in proportion to breadth and thickness, as to fail by flexure; for if one pillar be, in all its lineal dimensions, r times greater than another, its strength will be r^3 times as great on account of its increase of thickness, r times on account of its increased breadth, and less by $\frac{1}{r^2}$ times on account of its increase of length. It will thus be

$$\frac{r \times r^3}{r^2} \text{ or } r^2 \text{ times as strong.}$$

So that it would appear that the first circular and elliptical experimental tubes, constructed of very thin plates, were too hastily condemned from the unexpected failures in the top by buckling: had similar tubes been constructed of larger dimensions, then the strength of the top to resist buckling would have been as the square of the lineal dimensions of increase. Indeed, the curvature in the transverse plane of the top of these tubes was favourable to their resistance to buckling—a flat plate in a rectangular tube would have yielded with less strain.

252. As to the experiments given in line XX., the pressure per square inch has been set down as given in the Report, Appendix AA, p. 161; but Mr. E. Clark remarks, "that the bar was practically crushed with about 16 tons per square inch, though it carried considerably more." In the Table, § 239, it has, therefore, been given in the comparison with cast iron at that figure. There must always be a difficulty in deciding precisely on the point of greatest resistance in wrought iron pillars exposed to great compression, no sudden fracture indicating the exact weight by which its strength is overcome, as in cast iron under similar circumstances. It is for this reason, no doubt, that in the tabulated results of wrought iron the column which in cast iron is headed, Breaking Weight, is with wrought iron headed, Weight of greatest Resistance.

253. If we reduce the strength of the several plates in the ratio of the breadth to the thickness, we shall obtain the strength of a square pillar as deduced from that particular

plate; and, comparing those of the same length and different thickness, we obtain the power or index of the side of the square to which the strength is proportional. An average of these gives 3.59, very nearly identical with that given in page 231, § 235.

If plates have a given weight or quantity of iron, and the length or height be also constant, then their strength to resist compression varies as the square of the thickness; for, since $b \times t$ is constant, and therefore $b \propto \frac{1}{t}$, which last, being substituted in $\frac{bt^3}{l^2}$, gives $\frac{t^2}{l^2}$.

These results as to the laws of resistance of plates have been obtained from such as were bearing not more than from 1.076 tons to 8.923 tons per square inch of section. Up to about 9 tons per square inch we may conclude that the laws now given obtain; above that crushing begins to take place, causing a deviation from these laws; and the more so as the pressure increases, as will be evident in the experiments on rectangular tubes, pages 244 to 247.

254. As an illustration of the advantage of thicker plates in resisting compression, though not as a proof of the law of increase of strength above given, the following experiment of Mr. E. Hodgkinson may be quoted:—

A rectangular tube, 30 feet in clear span, depth 23.75 inches, breadth 15.3 inches, and having a top plate 0.272 inch thick throughout, failed with a weight of 22.8 tons. A plate of 0.437 thick was now substituted for about 15 feet in the centre at the top; it now required a weight of 32.5 tons to overcome its resistance, and with this load it seems to have failed at about 18 tons per square inch, and by tearing asunder from a rivet at the bottom; an addition of about 1½ cwt. of iron to a tube weighing 23 cwt., that is 6.5 per cent. of material, increasing its strength 50 per cent.

255. We have, however, yet further resources at our command, to obtain an effectual resistance to this dangerous buckling. When failure takes place in any wrought iron pillar, or the top of any ordinary tube, by actual crushing, it will be found to have sustained little more than about 16 or 18 tons per square inch of transverse section, and no alteration in its manner of construction would be of any avail. But, should it fail by buckling, its breaking strain would fall short

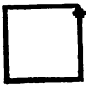
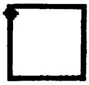
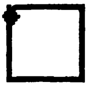
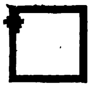
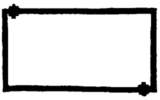
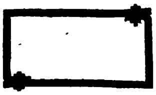
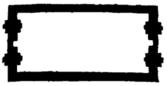
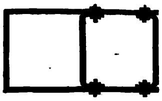
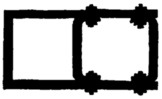
of this quantity; and any contrivance or method of construction which would give it rigidity, and prevent buckling, would bring the structure nearer to this limit of crushing.

Even at the very time of making the experiments on the greatest resistance of plates of wrought iron, it was seen that it would be necessary in those great girders, for the sake of which all these inquiries were originally instituted, to throw the iron both at the top and at the bottom into some form, either as a hollow cylinder or rectangular hollow tube, different from the simple plate, the necessary thickness of which would give rise to many practical inconveniences Thus, in the large span of the Britannia Bridge, the width of the top being 14.75 ft., and requiring 648.25 square inches in its transverse area at that part, it is evident that the thickness, as one plate or collection of plates riveted together, would be about 3.7 inches.

The advantages arising from the disposition of these plates in a cellular, instead of a solid form, are not alone practical, as affording means of escaping the difficulties of uniting them so as to act as one piece with full efficiency, but even in a more important point of view, as giving them a mechanical advantage in resisting flexure. There is an exact analogy between the benefit derived from this disposal of solid plates of wrought iron in the form of hollow cells, on the one hand, and that which has been already shown in §§ 150 to 152, to follow from changing a solid into a hollow cast iron column ; in both cases the material is taken from the central part about the neutral axis, where it has no mechanical advantage or leverage in opposing flexure, and added on in a position where this is attained in the highest degree which the assigned external dimensions of the cylinder or cell permit.

256. The following Table, pp. 244–247, gives the results of numerous experiments on tubes of wrought iron, of different external dimensions, from 4 to 8½ inches square, with the intermediate rectangular form 4 by 8 inches, and of varied thickness, from about $\frac{1}{30}$th up to ¼th of an inch ; the transverse areas ranging from 0.5 to 8.5 square inches. The lengths, which were chiefly 10ft., were also varied by cutting off the injured portions of the longer tubes, so as to give in most cases submultiples of 10 feet.

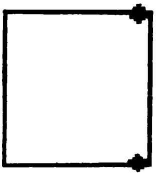
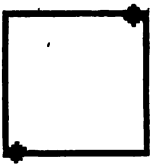
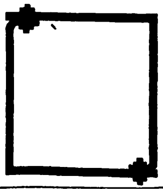
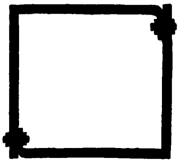
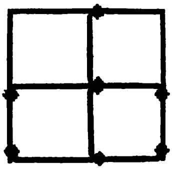
257. Table, *showing the Resistance of Wrought Iron Tubes or*

	Weight of Tube.		External Dimensions.	Thickness of the Plates of Tube.	Form of Section of Tube.
	Lbs.	oz.	Inches.	Inches.	
I.	21	2½	4.1 × 4.1	0.03, nearly.	
II.	43	14½	4.1 × 4.1	0.06, nearly.	
III.	60	6½	4.25 × 4.25	0.083	
IV.	80	6	4.25 × 4.25	0.134	
V.	65	8	8.175 × 4.1	0.061	
VI.	231	0	8.5 × 4.75	0.264	
VII.	233	0	8.4 × 4.25	.26 and .126	
VIII.	82	0	8.1 × 4.1	0.059	
IX.	290	0	8.5 × 4.75	0.25, nearly.	

Cells to a Force of Compression in the Direction of their Length.

Area of Section of Tube.	Weight at which buckling perceptible.	Weight of greatest Resistance.	Weight per square inch buckling perceptible.	Weight pr. square inch of greatest Resistance.	Lengths.	
Square Inch.	Tons.	Tons.	Tons,	Tons.	Ft.	In.
0.504	.. ..	2.47 2.79 2.79	.. ..	4.90 5.14 5.54	10 5 2	0 0 6
1.020	.. ..	8.77	.. ..	8.60	10	0
1.484 1.52 1.52	13.08	16.68 16.00 18.60	8.81	11.24 10.53 12.24	10 5 2	 0 6
2.3947	20.68	23.08 24.80	8.63	9.64 10.36	10 7	0 6
1.532	5.90	10.40 11.91 10.89	3.85	6.79 7.24 7.11	10 7 2	0 8 4
7.326	.. ..	88.02	.. ..	12.01	10	0
6.89	44.60	92.2	6.47	13.38	10	0
1.885	16.70	19.50 20.29 18.42 21.89	8.86	9.88 10.76 9.77 11.61	10 7 3 1	0 7½ 8 7½
8.3466	.. ..	.. ..	.. ..	not crushed with 11.12	10	0

257. TABLE, *showing the Resistance of Wrought Iron Tubes or*

	Weight of Tube.	External Dimensions.	Thickness of the Plates of Tube.	Form of Section.
	Lbs. oz.	Inches.	Inches.	
X.	91 8	8.1 × 8.1	0.06, nearly.	
XI.	162 0	8.37 × 8.37	0.139	
XII.	296 0	8.5 × 8.375	0.2191	
XIII.	333 0	8.5 × 8.4	0.245 and 0.238	
XIV.	157 4	8.1 × 8.1	0.0637	

Cells to a Force of Compression in the Direction of their Length.

Area of Section of Tube.	Weight at which buckling perceptible.	Weight of greatest Resistance.	Weight pr. square inch buckling perceptible.	Weight pr. square inch greatest Resistance.	Length.
Sqr. In.	Tons.	Tons.	Tons.	Tons.	Ft. In.
2.070	7.10	12.30 12.29	3.43 more observable with 4.27	5.93 5.94	10 0$\frac{3}{8}$'' 7' 8''
4.9262	36.82	44.8	7.47	9.10	10 0
7.7367	.. ..	88.82	.. ..	11.48	10' 0
8.4665	.. ..	not crushed with 100.8 tons.	.. ..	not crushed with 11.86	10' 0
3.551	25.28	31.28 36.62 36.79 38.29	7.12	8.81 10.31 10.36 10.78	10' 1'' 9' 10'' 5' 0'' 2' 4''

258. The ultimate object of this series of experiments was to determine the best form of transverse section in the top of the proposed bridges, both as to the size of the cells, and the thickness of the plates to be used in forming them, and thus enable the engineer to decide upon the details of the design of that part exposed to compression. The more immediate object was to ascertain the value of each particular form of cell experimented upon, and determine its resistance per square inch of section. For this purpose, fourteen different tubes were constructed, all ten feet in length, and differing in thickness of plates and dimensions.

The four inch square cells had a thickness, respectively, of $\frac{1}{32}$nd, $\frac{2}{32}$nd, $\frac{3}{32}$nd, $\frac{4}{32}$nd of an inch nearly, for to these fractions the decimals given in the column of "Thickness of Plates," nearly correspond, one thirty-second being 0.03125.

The rectangular cells had two thicknesses, nearly the $\frac{2}{32}$nd and $\frac{8}{32}$nd of an inch, both in those with and without the division plate uniting internally the middle points of the longer sides of the rectangle.

The cells of eight inches square had about $\frac{2}{32}$nd, $\frac{4}{32}$nd, $\frac{7}{32}$nd, $\frac{8}{32}$nd of an inch in thickness; and the last, having a thickness of $\frac{2}{32}$nd of an inch, was divided by two internal plates into four square cells.

The lateral dimensions of the cells was so great compared with their length of 10 feet, the ratios being $\frac{1}{30}$th and $\frac{1}{15}$th respectively, that they were not destroyed by flexure as in long slender pillars, but either by buckling or absolute crushing: if by the latter, we should expect that the strongest possible form and proportions would give about 16 or 18 tons per square inch, and whatever deficiency may be observed in this respect must be attributed to buckling having occurred first.

259. Although these experiments were too few in number and in variety of dimension to enable us to deduce the general laws of resistance of cells, yet several important practical deductions may be gathered from them—namely, as to the effect of form or size of cells, as to the value of thickness, and as to the influence of length on their greatest resistance.

We at once perceive that, as regards form, every other circumstance being the same, the large square cell with thin plates is the weakest; thus, comparing lines II. and X., the strength per square inch of the cell 4.1 inch square is 8.6 tons, and for that 8.1 inch square it amounts only to 5.9 tons, the thickness in each being 0.06, or $\frac{1}{16}$ of an inch. The next weakest form is the rectangle line V., which, with plates of the same thickness as the last comparison, had a strength of 6.79 tons per square inch, increased, however, fully 50 per cent., or up to 9.88 tons by a division plate across the centre, as in line VIII. The large square, divided into four smaller squares by two division plates, as in line XIV., gave still better results when tried the second time with a length of 9 feet 10 inches; for we find that it sustained, with the same thickness of $\frac{1}{16}$th inch, about 10.31 tons per square inch.

The Table below, arranged from § 257, shows the value of thickness on the ultimate strength per square inch, when read in the vertical lines; and the effect of form and size with equal thicknesses, in the horizontal lines.

§ 260. TABLE, *showing the comparative Strength of Rectangular Tubes, as depending on Thickness and Dimension of Sides, all* 10 *ft. long, except line* 5.

	Nos. of the lines of § 257 compared.	Tube 4 inch square.		in. in. Rectangle 8 by 4.		Tube 8 inch square.	
		Thickness of Plates.	Strength per square in.	Thickness of Plates.	Strength per square in.	Thickness of Plates.	Strength per square in.
1	I.	0.03	4.9	"	"	"	"
2	II. V. & X.	0.06	8.6	0.061	6.79	0.06	5.9
3	III.	0.083	11.24	"	"	"	"
4	IV. & XI.	0.134	9.64	"	"	0.139	9.1
5	XIV.	0.0637	10.31	"	"	"	"
6	XII.	"	"	"	"	0.219	11.48
7	VI.	"	"	0.264	12.015	"	"

261. These results, remarks Mr. Hodgkinson, are very singular: they show that, in the *crushing* of rectangular tubes, the strength, instead of being nearly as the third power of the thickness, as was found to be the case with plates, in § 248, is such that, to double the strength, four times, instead of $\frac{1}{4}$th, § 250, the thickness of the metal, is required:

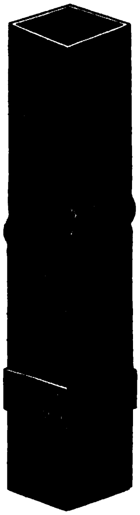
	Lines.		Thickness.		Tons.
thus—	1 4	Squares 4 × 4	0.03 0.134	give strengths	4·9 9·64
and	2 7	Rect. 8 × 4	0.061 0.264	give strengths	6·79 12·015

It also appears that when the thickness of the plates of the tubes is the same, the strength of the smaller ones is greater than that of the larger; as may be seen from the results in each horizontal line. It likewise appears that in square cells, compressed to a high degree, the length has little influence; for tubes 10 ft., 5 ft., and 2 ft. 6 inches, bear all nearly the same pressure per square inch. None but very thick plates resisted with 12 tons per square inch.

Thus, although the experiments on rectangular cells were too few in number and variety to establish the law of their strengths, they yet afforded practical suggestions of the highest value; thus it was determined to diminish the proposed dimensions of the cells in top of these great bridges or girders, which had been designed of a large size (about 3 ft. square); they were therefore constructed with the smallest sides which could be adopted; and it is a useful and curious illustration of the many circumstances which must be taken into account by engineers in the details of their designs, to find that the minimum was regulated by the consideration that each cell in the bridges must be capable of access to the men engaged in the periodical painting and scraping of the structure in future times. They were finally designed about 18 inches high, and 21 inches wide.

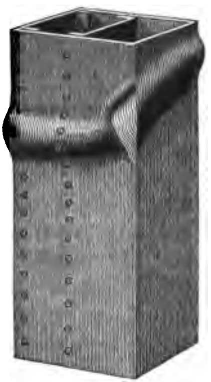
262. The method of failure is shown in the wood engravings attached—the first being that of the pillar, 2 feet 6 inches long, in line I., the thickness of which was 0.03 inch, and its transverse area 0.504 square inches, which finally yielded by buckling with 5.54 tons per square inch. It was cut down from a tube of 10 feet in length.

The second, page 251, represents the failure of the cell in line VIII., which was cut down, after successive experiments, from 10 feet in length to 1 foot 7½ inches, the thickness being nearly 0.06 inches and the transverse area 1.885 square inches. It yielded by buckling with about 11.6 tons per square inch.

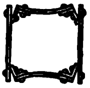


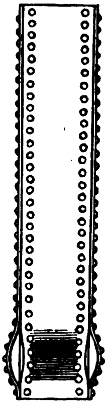
263. One experiment further must be recorded, in addition to those given in Table, § 257, deserving of notice both for its immense scale and for its satisfactory results. It is that by Mr. Fairbairn on a square cell nearly the dimensions of those of the intended structure, namely, 18 inches square,

with plates ½ inch thick riveted to angle irons, and 8 feet long It was placed vertically under a powerful hydraulic press, which is described as being of the best construction, having four distinct cylinders and rams, the pressure being indicated by an exceedingly sensitive lever with a carefully graduated scale, on which the strain during the different stages of the experiment was recorded. The sectional area is given as 50 square inches, and the pressure of ultimate resistance at 680 tons, so that the resistance per square inch was 13.6 tons.



The engravings below show, in elevation and in sectional plan, the manner of failure, which was by the buckling of three of the sides near the top. In the words of Mr. Fairbairn—" A short time before the whole amount of pressure (690 tons) came upon the tube, it was progressively yielding to the strains. It ultimately became distorted by the buckling of three of the sides, as exhibited in the drawing." The reduction in length is given as 0.16 inch by 665 tons, which is rather greater than the 10000$^{\text{dth}}$ part of the length for every ton per square inch.





This is one of those great full-size experiments which we never meet with out of England: no law of strength as dependent on change of dimensions is in such required; the model is as large as the structure itself. It only remained to surmise that some increase of the "greatest resistance" would probably accrue when, as in the actual structure, a number of cells are rigidly connected side by side, and consequently all lateral deflection or buckling counteracted in the horizontal plane: and also that, in the vertical plane, the firm attachment of the sides to the outermost cells would tend to maintain the whole of the top in place. The final experiments on the large model tube, 75 ft. span, gave, by calculation, a resistance to compression in the top cells of 14.8 tons per square inch when it failed. This is a vast improvement on the results of the earlier experiments on the wrought iron model beams, 37 to 30 ft. span, which, we have seen (§ 243), yielded with about 4 or 5 tons per square inch of compression: thus, although the dangerous buckling of the plates shows itself even in the last great experiment of Mr. Fairbairn, yet its evil effects are so far mitigated by the rigidity of the cellular form, that we can nearly approach the ultimate resistance of the material; a result which must be esteemed a compensation for the labour and expense of these experiments, although their number and variety was less than could be desired.

It is, however, to be regretted that the hint we obtain from the peculiar form and from the resulting strength in line VII. was not followed up: this it was which gave the highest result to be found in the Table, namely, 13.39 tons per square inch, the angles being joined on the short sides of the rectangle by thin strips, only about $\frac{1}{8}$th of an inch, that is, half the thickness of the remainder of the cell, which was $\frac{1}{4}$th of an inch. We may well infer from this, that in rectangular cells, acting as pillars, the chief resistance to distortion of form under great pressure resides in the angles. It would also have been interesting to have compared the pillar in line XII. or XIII. with another of exactly the same total transverse area and length, but with much thicker plates at the angles, united by riveting four thinner strips on each side. If we examine the transverse sections of the cells in the top of the Britannia Bridge, Plate XIII., fig. 159, we may see that the cover plates outside and the angle irons in the interior angles give an approximation to such a form. It might also be surmised that a circular cell or tube of moderate dimensions, and of given thickness and section, —a figure which is identical in every vertical plane passing through the axis,—would be a stronger form than the same plate with a rectangular section, in which the resistance to crippling depends solely on the four angles.

264. As regards the effect of length on the resistance of cells as pillars, it appears, that, instead of the strength varying inversely as the square of the length, as in long flexible plates, it had but little ultimate influence in those square and rectangular tubes which were compressed to a high degree. Thus in line III., page 244, we have:—

Length.	Breaking Weight.
10 ft.	11.24 tons per square inch.
5 ft.	10.53 ,, ,,
$2\frac{1}{2}$ ft.	12.24 ,, ,,

And again, in line V. we have:—

Length.	Breaking Weight.
10 ft.	6.79 tons per square inch
7 ft. 8 in.	7.24 ,, ,,
2 ft. 4 in.	7.11 ,, ,,

These tubes were, as to length, in the same circumstances as short solid specimens of the same material, in which, within the limits of a height about equal to the diameter and 4 or 5 times the diameter, the resistance to crushing is constant, no flexure whereby part of the transverse area would be thrown into tension being possible.

265. It now remains to give the experiments on hollow cylinders as pillars. In these the transverse area ranged from about half a square inch to 2½ square inches, and the corresponding external diameters from 1½ inches to nearly 6½, which give $\frac{1}{80}$th and $\frac{1}{20}$th of the length, respectively.

§ 266. TABLE, *showing the Resistance of Cylindrical Tubes of Wrought Iron to a Force of Compression in the Direction of their Lengths, the ends being turned perfectly* flat.

Nos. of Experiments.	Diameters.		Thickness.	Area.	Weight of greatest Resistance.	Pressure on square inch of greatest Resistance. Lengths.	Lengths.	
	External.	Internal.						
	Inches.	Inches.	Inches.	Sq. Inches:	Tons.	Tons.	Ft.	In.
I.	1.495	1.292	0.01	0.4443	2.918	6.55	9	11
	do.	do.	do.	do.	6.19	13.92	5	0
	do.	do.	do.	do.	6.78	15.28	2	6
II.	1.964	1.755	.. ..	0.614	6.32	10.35	9	11
III.	2.49	2.275	.. ..	0.8045	10.69	13.29	9	11
IV.	2.35	1.865	.. ..	1.605	15.41	9.60	10	0
V.	2.34	1.91	0.215	1.4353	14.21	9.90	10	0
VI.	2.995	2.693	.. ..	1.349	16.68	12.36	9	11¼
	do.	do.	do.	do.	18.80	13.30	7	6
	do.	do.	do.	do.	23.60	16.69	2	4½
VII.	4.05	3.772	.. ..	1.7078	21.08	12.34	9	11¼
VIII.	4.06	3.75	0.150	1.901	22.28	11.71	9	11¼
IX.	6.366	6.106	0.1298	2.547	40.80	16:02	10	0
X.	6.187	.. ..	0.0939	1.799	26.82	14.91	10	0
	do.	do.	do.	do.	30.80	17.12	5	0
	do.	do.	do.	do.	33.22	18.46	2	6

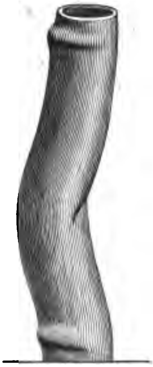
267. It appears at once, by comparing the columns in the above Table headed, "Pressure per square inch of greatest Resistance," with the same in the Table, pages 245, 247, of rectangular form, that the cylindrical tubes give higher results, and that the surmise at the end of § 263 is borne out by these experiments.

The columns with the smaller diameters may be expected to bend as in long columns, and follow, in some degree, the laws that they are subject to—the resistance, where the pressure is small, being inversely as the square of the length nearly. On the contrary, in tubes 6 inches in diameter, and only 10 feet long, flexure cannot take place, except in a very small degree; and therefore the resistance will be nearly constant, and of the nature of crushing or wrinkling, as was the case in all the experiments on the compression of rectangular cylinders. In these we have seen that the resistance was nearly the same, whether the tube was 10 feet long, or any part of that length.

"In experiments on the longitudinal resistance of cylindrical tubes of small and large diameter, we have the operation of two distinct laws exhibited. Thus, in line I., we have the resistance increasing as we diminish the lengths to one-half and one-fourth, from 6.55 tons to 13.92 and 15.28 respectively, which, although not proportioned to the inverse square, show a very great increase of strength as the length is diminished. In the tube 6 inches in diameter, line X., we have also three lengths, the second and third being the half and the fourth of the first, which was 10 feet long, the respective resistances being more nearly equal than in line VI., in which the external diameter was 3 inches, or $\frac{1}{40}$th of the length of 10 feet. For, in line VI. with about 3 inches diameter, and lengths of 10 feet and 2 feet 6 inches, we have the extreme strengths 12.36 and 16.69 tons per square inch, which differ by 4.3 tons; while in line X., with 6 inches diameter, the extremes are 14.91 and 18.46 tons per square inch, with lengths also 10 feet and 2 feet 6 inches, respectively, which differ by about 3.5 tons: and hence the inference very fairly suggests itself, that, if the diameters had been yet further increased, we should have had the strength independent of the length."

268. The wood engraving shows the method of failure of the cylinders generally. This particular one failed by buckling and flexure: it was 2 ft. 4.3″ long, and 4 inches in external diameter, having a thickness of about $\frac{1}{4}$th of an inch.

The diminution of length was about 0.58 inch with about 55 tons; with 60.8 tons the greatest resistance was overcome.



269. It was a favourite idea of Mr. Hodgkinson that the circular cell should be adopted in the tubular bridges, at least in the parts subjected to compression; but the practical difficulties of uniting them to each other and to the sides, and their inaccessibility for examination or painting when once erected, precluded their adoption. It was reserved for Mr. Brunel, in his Chepstow and Saltash Bridges, to take advantage of whatever gain this form may promise; the top, or compressed member, in the former being a tube 9 feet in diameter, slightly cambered; and in the latter an ellipse, 16 feet by 12, with the larger diameter horizontal Plate XIII., fig. 159.

PRACTICAL APPLICATIONS AND EXAMPLES—WROUGHT IRON IN COMPRESSION.

270. The only object of the experiments on vertical tubes of wrought iron given above was, as has been already mentioned, with the view of deciding upon the proper proportions of the great beams then under consideration, from the transverse strains in which a severe compression was indirectly produced in the upper part. Notwithstanding this limited aim, the experiments themselves are evidently to be placed amongst the results of material under direct compression, the tests applied being exclusively of that character, and their application consequently extending legitimately to the assigning of the dimensions of solid and hollow pillars of this material under a vertical force. Examples are not wanting in recent civil and mechanical engineering in which wrought iron has been employed as a pillar with advantage in special circumstances, although, speaking generally, it cannot be said to be fully recognised with this application in the constructive arts.

271. Lighthouses have within a recent period been constructed on a foundation of Mitchell's screws bearing up solid wrought iron pillars, to the heads of which the superstructure is subsequently connected above the ground line; it is thus that

invaluable lights have been placed on sandbanks, and other situations where the construction of the usual tower of masonry would have been utterly impossible.

The following account of that constructed for the Spanish Government by Mr. John H. Porter is abridged from a paper read by him before the Institution of Mechanical Engineers, and published in their Proceedings for 1861. The general elevation is shown in Plate XIII., fig. 160. It was constructed on the island of Buda, at the mouth of the Ebro, and is intended to be visible 20 miles distant out at sea. It will be observed from the dimensions figured that the screws were intended to have been driven 30 feet into the ground.

"The Buda lighthouse is erected on the margin of the sea, not actually in the water, but so close, that in very stormy weather, its base may be washed by the waves and spray. [The Mediterranean, it will be remembered, is of an almost unvarying level.] Owing to the sandy nature of the ground, Mitchell's screw piles are adopted for the foundation: eight of them are arranged in an octagon of 56 feet diameter, and a ninth is placed in the centre. Upon the top of the wrought iron piles, starting at about 3 feet above the level of the sea, is an octagonal pyramidal structure of open iron work, 150 feet high from the top of the piles to the level of the lantern platform at the summit, as shown and figured in the general elevation.

"From the centre pile rises a hollow cast iron column to a height of 31 feet, from which point radiate the horizontal beams that support the platform of the dome-shaped dwelling house. To protect this platform from the violence of the wind, an inverted cone is constructed beneath, serving also as a receptacle for stores. From the house platform, rising through the dome to the lantern platform at the summit, is a wrought iron cylindrical tower, enclosing the winding staircase by which the keepers ascend to the light. From the eight external piles rise eight wrought iron pillars, converging from an octagon of 56 feet diameter at the base to one of 9 feet 10 inches diameter at the summit. These supports are united horizontally at ten points in the height of the structure by sets of horizontal framing adapted for resisting compression or extension, the intersection of which with the uprights forms thus a series of quadrilatral spaces in the sides of the octagonal pyramid; and these spaces are crossed diagonally by round tie rods, which are united in a centre ring, to admit of being screwed up to a state of tension. Other tie rods radiate obliquely from the central tower to the external uprights, each fitted with a right and left handed screw coupling, and are attached to the tower midway between the tiers of horizontal framing; eight of them ascend, and eight descend to the junctions of the framings with the corner pillars. The eight ascending rods thus form, in connexion with the horizontal framing, under compression, a system of trussing analogous to that commonly employed in the interior of gasholders, and so distribute their portion of the central load upon the external supports. By this arrangement the weight of the central tower and stairs is sustained at intervals in the height of the structure, by the eight corner pillars, and but slightly by the dome or by the centre pile. The set of eight descending rods assists with other portions of the structure, in preventing any distortion of the general fabric.

"The total weight of the superstructure, including the lantern, is about 170 tons, of which about 40 tons are upon the centre pile, leaving from 16 to 17 tons to be borne by each of the eight corner piles. The lantern and lighting apparatus weighed about 10 tons."

272. The following statement is given by Mr. Mitchell as to the experiment by which he determined a practical rule, giving the requisite bearing area of the screw in proportion to the insistent weight,* in the case of the Maplin Sand lighthouse, constructed on screw piles at the mouth of the Thames.

A jointed rod, 30 ft. long, and 1¼ in. in diameter, having at its foot a spiral flange 6 in. diameter, was moved round by means of cross levers, keyed upon the boring rod; and upon these levers, when the screw was turned to the depth of 27 ft., a few boards were laid, forming a platform sufficiently large to support twelve men. A bar was then driven into the bank at some distance, to supply a bench mark or point of reference, its upper surface being brought to the same level as that of the boring rod. Twelve men were then placed upon the platform, to ascertain if their weight, together with the apparatus—in all about one ton—sufficed to depress the screw. After some time the men were removed, and the level was again applied; but no sensible depression of the screw could be observed. The inference from this was, that, if a screw of 6 in. diameter could support one ton, another having one ft. diameter was capable of supporting at least 4 tons, the comparative area of their surfaces being as the squares of the diameters; and hence is derived the rule—"Four times the square of the diameter in feet is equal to the bearing power of the screw in tons." But, as a continuous surface possesses a much greater sustaining power than the same area in detached portions one foot in diameter, we find the number five in use in practice, as a constant that can be employed with perfect safety in a foundation of sand.

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* It was originally undertaken to determine this point.

273. The screws for this structure are 4 ft. in diameter, and are attached to wrought iron solid cylindrical piles, 8 in. in diameter, and originally intended to enter 30 ft. into the sand, as shown in the general elevation, fig. 160, and at a larger scale, fig. 161. The piles lettered A are solid, of hammered iron, forged in one length; the upper ends, as shown in the vertical section, fig. 162, are reduced to 7 in. diameter for a length of 22 inches, at which point a collar is welded to the body of the 8 in. pile, on which rests the cast iron cap lettered B. In Fig. 162 *a* is shown the manner in which the wrought iron pile is attached below to the cast iron screw: there is a collar forged on the pile, forming a shoulder or stop, which bears upon the top of the screw; and below this shoulder the pile is reduced on two opposite sides, forming a kind of tenon, which is dropped into a corresponding socket cast in the screw, so that it must turn with the pile: a key of wrought iron passes through the opposite sides of the socket and the flattened portion of the pile; it is very accurately fitted, and left flush on the outside, having a slight taper or wedge form.

The centre pile, with its cast iron cap in vertical section, is shown in fig. 166; and the cast iron column springing from it, with the connexion of the radiating girders of the horizontal framing and of the cone below the house platform. The junctions of the cast iron column are all bored, turned, and faced in the lathe. The head of the pile above the shoulder is reduced to 7 in., as in the eight external ones, but for double the height, or 44 in.

274. Above the level of the caps of the eight outer vertical piles, which are attached to the screws, and rest upon them, rise eight wrought iron pillars, the form of which is shown in section, figs. 170, 171.

The lowermost part of the pillars, fig. 170, consists of a bar 12 in. by one inch, with angle irons of 4 in. by 5 in., and ¾ in. thick, united to it by one inch rivets, spaced 6 in.

apart. This section is carried up to a height of 5 feet above the level of the house platform, from which point the smaller section, fig. 171, is employed. This differs from the former only in the centre bar, which is reduced to 9 in. in width. The transverse section of the lower part is 24, and the upper 21 square in., which is far more than necessary for resisting the forces which compress it in the direction of its length. There is thus no portion of the pillar of a less thickness than ⅝ in., and all parts of it can be conveniently inspected after erection, and carefully scraped and repainted when necessary; while the flanged structure of the pillar itself, as derived from the angle irons, greatly contributes to its stiffness in all directions. The unbraced length of the pillar was about 16 ft. 4 in.; which is the vertical distance between the several horizontal framings.

275. In addition to considerations of general construction and detail, so as to secure the rigid connexion of all the parts of the structure, it was necessary to have regard to simplicity, convenience, and safety in the process of erecting abroad. With this object, the jointing of the several lengths of the external corner pillars is arranged so as to admit of each stage or tier of horizontal framework being permanently attached to the pillars at a distance of 5 feet below the centre of the joint of the middle plate; the angle irons have their joints 6 ft. apart, one being 3 ft. below, and the other 3 ft. above the meeting joint of the end of the middle plate; angle iron cover plates, 3 ft. long, are riveted over these joints, and a flat plate, also 3 ft. long, over the joint of the middle bar. The lower angle iron cover plate is consequently some 6 in. clear above the horizontal framing, each stage of which thus affords, with the addition of some scaffold boards, a most secure and convenient platform to place the materials on for proceeding with the erection of the succeeding lengths of the external pillar and central tower. The junctions of these parts of the structure can there be conveniently riveted and bolted, the diagonal tie rods attached, and their degree of tension adjusted with accuracy.

276. The central tower, already mentioned, rises from the house platform immediately over the cast iron column; it is 6½ ft. in diameter, and contains the circular stairs, and is formed of plate iron, ½ in. thick. Six plates in width form the circumference of the cylinder. The edges of the plates are butted on all sides, with cover strips on the exterior: those covering the horizontal joints are 6 in. wide by ⅝ in. thick, and those

of the vertical joints 4·in. by ⅜ in. thick. The winding stairs within the tower are of cast iron, and strung upon a central tube of wrought iron, 2 in. in diameter: a complete revolution of the stair is made in a height of 8 ft. 2 in., and this central tower was raised in cylinders of that length; and two such lengths correspond exactly with the spacing of each tier of framing and ties above the house dome.

As the work of constructing proceeds, a permanent and secure means of ascending and descending is provided—the small window openings admitting at the same time of a man passing through the side of the tower to the platform of framing next below it. This special advantage will, moreover, always exist for the inspection of all parts of the structure, affording the same convenience for painting when occasion requires.

In two wrought iron lighthouses, erected near the mouth of the Thames (the Mucking and Sea Reach), the pillars of the superstructure at the angles were formed of hollow cylinders of wrought iron, about 10 inches diameter, which were constructed of ¾ inch plates, bent round, and riveted together with a butt joint.

This method of construction had an advantage in point of appearance; but it is decidedly expensive, and almost impossible to paint inside. In the method adopted in the Buda lighthouse, described above, the entire surface is exposed to view, and accessible for painting and examination. A precedent for using cast iron in the main pillars of a lighthouse is afforded by the screw-pile lighthouse erected by the Government of the United States on the Florida Reef: in this, the only other structure at all comparable in dimensions with that at Buda, the principal supports consist of cast iron tubes. It was, however, considered that there must be some insecurity in employing so brittle a material, and that it was less fitted to undergo the vibration or tremour which ccasionally, and perhaps to a certain extent generally, will pervade a structure of this character and of these dimensions; added to which was a consideration of the inconvenience that would result from the breakage of any of the cast iron tubes in transit, in transhipment, or in erection abroad. Wrought iron having thus been determined upon as the material to be used, the form in which it could best be employed next came under consideration. Several advantages attending on that which has actually been adopted have already been mentioned, and it may be added that the connexions of the several lengths of a pillar with each other, and also with the horizontal and other framing, would have been attended with more expense and less simplicity in detail when in the form of a hollow cylinder than that shown in figs. 170, 171.

277. The diagonal bracing is on an excellent system, and experience has pointed out that some such provision is essential in these structures: thus the Maplin Sands lighthouse was originally designed without diagonal stays; but when erected it was found that, though strong enough vertically, the whole rocked and twisted round horizontally, when a moderate force was exerted at the top of any one of the pillars at regular intervals, so as to get into time with the vibration of the structure; such a motion might have proved fatal to its stability, and diagonal stays were therefore subsequently added, similar to those in the lighthouse at Buda, which entirely corrected the defect.

In the design for the Bishop's Rock lighthouse, near the Land's End, constructed of a framework, with cast iron pillars at the angles, there was no diagonal bracing for the first 12 feet high; so that in a storm, shortly after it was erected, the whole lighthouse had twisted round, and the pillars broke off at the base: had it been properly stiffened with diagonal stays, starting from the very foundation, it might have been a most satisfactory and secure structure. It was, however, rebuilt of stone, at a greatly increased cost.

278. At Buda, in consequence of the density of the sand, and size of the screws, it was found impossible to get the piles down to the intended depth, so that additional bracing was introduced, about 6 ft. below the pile heads; it was similar in construction to the lowest bracing in the original design, and of the same material, namely, puddled steel, by the use of which a saving in weight of 25 or 30 per cent. was effected, and advantage taken of its resistance to corrosion, in which it was deemed superior to the ordinary plate iron. These two deep and rigid framings must secure the piles against any lateral stress, and relieve any single pile which, from unexpected weakness in its foundation of sand, might stand in need of such assistance.

Moreover, while the lighthouse was in course of erection, the River Ebro began to change its course, and continued to do so, until at length the structure stood and stands in mid-channel, instead of on dry land. As the river scoured away the sand from around the piles, it became necessary to pitch in stone to the extent of a small island. This separation from the mainland compelled other additions for dwellings, for water supply, &c. &c., as the requisite supply of stores might be cut off during any continuance of bad weather. The tower has been found perfectly steady and rigid in a heavy gale of wind and rough sea.

279. The drawings, figs. 178, &c., represent an application of wrought iron pillars to support the successive tiers of boxes and galleries in a Theatre, and is equally applicable to any structure having public assemblies for its object. In such structures it is obviously of importance that the unavoidable introduction of vertical supports should as little as possible interfere with the view of the stage, pulpit, or platform, as the case may be; and for this purpose a solid wrought iron shaft, when the height, as in the case before us, is not required to be considerable, would seem to be the most suitable material of all at our command, and also the best form; timber of sufficient strength being more bulky, and subject to the risk of fire; and cast iron, on the other hand, if of equal diameter with that in the drawing, would be inferior in its ultimate vertical resistance, and also liable to sudden fracture from any accidental transverse blow. The pillars in the drawing, which are 4 inches and 3½ in. in diameter, and about 10 ft. and 8 ft. 9 in. in length, respectively, can be purchased as rolled in one piece without any welding. The only difficulty in applying such a uniform cylinder in practice would seem to be the want of sufficient area at the base, and at the upper end: this, however, is efficiently provided by the adjunct of a cast iron base plate and capital, the wrought iron pillar being turned at each end, and the castings bored out to fit them: the neck of the wrought iron pillar is reduced for a length of 6 in. to a diameter ½ an inch less than the other part of the shaft.

The lower base plate, which is circular in form, and 14 inches in diameter, is bolted down upon a timber sill, 14 in. wide, and 6 in. deep, by four screw bolts, one inch in diameter. It is shown in elevation, fig. 180; in plan, fig. 181; and in vertical section, fig. 182, the last pointing out the depth which the wrought iron shaft enters the base plate.

The capital is of an ornamental character, and of an identical form in each tier, but smaller above, as the wrought iron shaft itself decreases from 4 in. diameter in the lowermost range to 3½ in the upper; a piece of deal plank, ½ in. thick, and 7 in. square, is placed centrally on the upper surface of the capital; and on this the angle irons, forming the lower flanges of the three wrought iron girders, which intersect over the centres of the pillars, rest; this economizes, saving both the planing of the upper surface of the capital, the timber under pressure adapting itself to small inequalities, and also dispensing with the countersinking of the rivet heads, the form of which may be cut out of the timber with a gouge. The girders are fastened down by four screwed bolts, tapped into the capital, and passing through the slab of timber.

It will be perceived that the top plate of the projecting cantilever passes over the angle iron forming the upper flanges of the intersecting girders; between this and the bottom surface of the base plate of the second tier a half inch plate of timber is also introduced: the several views of this casting are shown in figs. 183, 184. The inclined directions of the vertical ribs, fig. 183, are for the purpose of supporting the intermediate girders in the curve of the front.

The bolts holding down this casting pass through all, and have a nut and washer below the angle irons of the upper flange of the longitudinal front girders. The form of the capitals of the pillars of the dress circle is, as the drawings point out, identical with that of the lower tier.

280. It will be observed that the weight borne by the base of the upper range of pillars is transmitted to the lower through the portions of the three girders which intersect in the line of their common vertical axis, and which thus supply the place of a connecting piece, the vertical supports resting upon the horizontal framework of girders, to which they are securely bolted—a design which is quite admissible in structures, the loading of which is not intended to be greater than it is ever found to attain in such places of public assembly described in this example. It would probably be ample to take one cwt. weight per square foot as the greatest load, including the gallery and seats, with a full audience.

281. The framings of Marine Engines, especially the oscillating and the overhead forms of construction for paddle and screw vessels, respectively, give occasion for the employment of solid wrought iron pillars with great advantage. The drawings, figs. 187, &c., show a pair of pillars of this material, part of the frame of a screw steamer constructed by the firm of Messrs. Courtney Stephens, and Co., Dublin. It will be perceived that the shaft of the pillars, 3½ in. in diameter is enlarged at each end, by welding on suitable pieces, so as to give shoulders whose external diameter is equal to 5½ in.; each of the pillars, which are all faced and turned in the lathe, passes in its upper end through an opening

in a projecting lug, cast upon the body of the condenser, and bored out and faced to receive it; the several pillars, having a screw cut upon the upper end, are firmly held by a hexagon nut, worked down to bear upon a washer laid on the upper surface of the casting; at the lower end it enters a projection, 9 inches in diameter, cast upon the bed plate, and bored out true to receive it; in this it is held by a tapered cotter, 3 in. by 1 in., passing through all, and very carefully fitted. The pillars are consequently adapted to act either as tie rods to hold the parts of the engines together, or as columns to bear their weight, which they may be called upon to do, either from the working of the ship in a seaway, or from the alternate motion of the overhead engine.

In such positions, the wrought iron pillar has great advantages over cast iron, among which are its lesser bulk, and more reliable character under such varied strains, and its admitting of the methods of fastening just described.

282. The difficulty of designing Railway Bridges is, in particular localities, very great, both with those passing over and under the line. The exigencies of providing the necessary breadth, the requisite headway, and of compliance with the many requirements of the Legislature, are often very trying to the skill of the engineer, and oblique and inclined directions of crossing generally present themselves to enhance the other troublesome circumstances. In the case of an over bridge—one in which the street or road is carried over the line of rails—the least headway is measured by the highest funnel of the engines used on the line, some few inches being allowed for the issuing blast; and in case of any super-elevation of the outer rails, from curvature of the line at the spot, this also must be attended to, as it in some degree raises the funnel: in many instances 14 ft. has been found sufficient, measuring from the surface of the rails to the soffite of the bridge.

The horizontal girder, placed at the height thus determined, is very generally used to carry the roadway, and it is evidently the depth of this girder, together with that of the material of the roadway, added on to the 14 ft., or whatever it may be, mentioned above, which, in the case of bridges over the railway, gives the limit to the vertical distance between the line of rails and the line of road.

The depth of the girder will, to a considerable extent, depend upon the span, and this for a double line is generally about 28 or 30 ft. Now, by placing an intermediate support between the up and the down line, the span over each of them, respectively, can be reduced to that sufficient for a single line of way, which is about half of that required by a double line; and the distance between the abutments is only increased by the thickness of the intermediate support introduced, which, in the case of masonry will be from 18 in. to 2 ft., and with cast iron columns probably 12 or 15 in. for the external diameter of the pillars adapted to this duty; but the strength and depth of the girder may be greatly diminished with a span of only one-half, and consequently the vertical distance between the level of the surface of the road and the level of rails to the same extent lessened.

283. The drawings, figs. 191, &c., will explain the construction of the bridge under Arklow street, on the Dublin and Wexford Railway, designed by Mr. William R. Le Fanu. The street, which was crossed obliquely, could not be altered in level; and from the nearness of the station platform, the level of the rails could not conveniently be lowered—it being, on the contrary, rather an advantage to place them as high as possible; hence the peculiarities of the design which was adopted to meet these circumstances. In it we find that the span for a double line has been divided into two several openings by a range of seven wrought iron pillars, placed between the up and down lines; these are formed of axle iron, 4 in. in diameter, and, being truly faced at each end, are dropped into a base plate below, and at the upper end carry a cast iron cap adapted above to bear the bottom flange of the wrought iron girders; these are formed of a vertical web, 10″ deep, and ½″ thick, the upper and lower flanges being constructed of angle iron, 3½ × 3½ × ½″, riveted on each side of the web: between each pair of girders, which are 5 ft. 3 in. apart, plates of wrought iron, about 3 feet wide, and ½″ thick, bent into a segmental curve, are placed, and simply rest upon the lower angle irons. The parapet wall is carried by a cast iron girder; and, as the depth here has no influence on the headway, or on the level of the street, it is unsupported in the centre, spanning both lines of way with a single opening. Wrought iron ties are passed from outside to outside in the line of the central pillars, and riveted at each column. The spandril spaces are filled in with concrete up to the line of the top of the curved plates and wrought iron girders, on the surface of which the road metalling is placed, making the total depth from the street to the tie rod 15 in., and from the street to the line of rails 15 ft. 3 in., which is

probably the least possible interval with the ordinary height of funnel of railway locomotives. In under bridges very ingenious designs for a minimum headway may be observed; but, as the public roadway cannot be divided, they afford no examples of the application of an intermediate line of pillars.

In arches of masonry the same end may often be obtained by placing one small flat arch over each line of way, in lieu of a large single arch over both lines, of greater rise and greater depth at the key. The relative alteration, or saving of vertical height may be as great in this case; but the structure now drawn and described gives absolutely the least vertical distance between road and rail.

284. Pillars of wrought iron have been advantageously employed in the construction of gas holders, and with great economy. That represented in figs. 199, &c., is taken from a paper by Mr. A. F. Wood, published in the "Transactions" of the Society of Engineers, 1864. Contrary to the usual practice, the columns which surround the gas holder at the Kennington-lane Station of the Phœnix Co. have been formed of plates of malleable iron riveted together. The gas holder is 160 feet in diameter, and 70 feet high, and has 16 columns outside its circumference, which carry on their inner faces vertical rails, serving in the usual manner to guide the ascent and descent of the holder. Each of the columns is 73 feet in total height, and 3 feet 8 inches diameter at the base, tapering to 2 feet 8 inches at the top; the plates are $\frac{3}{8}$ths in. thick below, and $\frac{1}{4}$th in. above. Each column was erected in one piece, and stands in a cast iron base held down by four long bolts, which are attached to a plate built into the masonry of the circular wall. The external mouldings of the columns are of cast iron; and, being formed of two pieces, which meet in a diametrical vertical plane, they are easily attached to each other and to the wrought iron column, by screw-bolts. The usual cast iron girders holding together the heads of the pillars have been superseded by wrought iron; here also, in the form of tie rods, 2 inches and 1$\frac{1}{2}$ inch in diameter.

Had cast iron pillars been employed, they would probably have been cast in three lengths, with internal flanges faced and turned in the lathe, and united by screw bolts on the inside; and, although having very little vertical load to sustain, the thickness of the metal could not have been less than $1\frac{1}{2}$ or $1\frac{1}{4}$ inches, whereas the plate iron was but $\frac{3}{8}^{\text{ths}}$, which more than compensates for its higher price per ton.

The main building for the Paris Exhibition, 1867, gives a fine example of wrought iron pillars: the great external oval gallery is covered by an iron roof, in form a circular arc; this it was determined to construct without any internal tie rods or trussing, which would have induced a severe outward thrust on the supporting pillars at their summit but, for the device of carrying them up outside all to the level of the crest or ridge of the roof, and connecting them together by external tie rods: a force is thus supplied in opposition to the horizontal thrust of the curved roof; the pillars have therefore evidently to resist a transverse strain as beams, arising from the circular roof, as well as to sustain its weight; with this object they are constructed of a rectangular form in the horizontal section, with the greater side in the direction of the thrust of the roof, and diminishing towards each end from the point of application of that force, as in a horizontal beam, the firm attachments at the base and the tie rods at the summit acting as abutments.

285. Many other applications of wrought iron must occur to the reader, such as the masts of ships, which are now sometimes made as hollow cylinders of plate iron riveted together and also more recently of steel plates in order still furthe to reduce the weight. An advantage is ingeniously taken o these lofty hollow cylinders to ventilate the lower decks through a lateral opening below, and a corresponding one at the top, the foul air passing up as through a funnel.

The great shear legs so requisite at every port and dockyard, for lifting in and out of place on board, masts, boilers, and heavy portions of marine engines, are also now generally formed of hollow plate iron cylindrical tubes; but both in these and in the case of masts the duty of resisting transverse strain so far exceeds anything they are called upon to support as pillars, that they must be left for description under the examples of wrought iron beams.

We may frequently observe wrought iron pillars, in the form of rectangular hollow tubes, supporting roofs with open sides at various workshops; but it is probable that the proprietors of these establishments, making use of plate iron as one of the materials of their regular business—such as boiler makers, iron ship builders, &c., have been influenced to adopt it for a vertical support, when making alterations or additions to their works, with the motive of keeping their hands together (especially in a time of slackness in trade), and putting them to work on a material which they thoroughly understand, and can manipulate with skill; so that we cannot point to such work as giving an example; it would probably have been cheaper to have used cast iron columns, but for the contingent advantages mentioned above.

DIRECT RESISTANCE OF TIMBER TO COMPRESSION.

286. **Timber.**—*The direct resistance of Fir and Oak Timber to a compressive force is about 2½ to 4½ tons per square inch respectively in short specimens, ends flat, carefully selected, very dry, and seasoned, and of the species most generally used in construction, and is proportional to the area of section in action.*

287. The experiments were made on short columns 2 in. in height, flat at the ends, and turned to be one inch in diameter. A very important addition to the usual line of experiments has been followed in these, which have been tabulated (§ 288), namely, obtaining the ultimate resistance of the specimens in two very different states: first, when ordinarily dry or fit for use; and, secondly, when, after being turned, they were kept drying in a warm place for a further period of about two months. The result is of great practical importance, as showing the good effect of seasoning or drying upon wood, and the great comparative weakness of wet timber, it having, with many different kinds, not half the strength of that which is thoroughly dry.

The following Table is arranged from that given by Mr. E. Hodgkinson, in which the mean results are set down, commencing with that species which has the highest resistance, and so on in order down to the lowest. The first column gives the results from a mean of about three experiments on each species, the woods being moderately dry, such as are employed in making patterns for castings. The second is derived from a like average, the specimens being, as has been mentioned, very dry:—

288. TABLE, *showing the Ultimate Resistance of short Specimens of Timber to crushing, ends* FLAT, *with the Resistance to Tension, from page 54, appended, in some of them.*

Description of Timber.	Compression: Tons per square inch.		Tension Tons per sq. inch.	Ratio of Compression to Tension.
	Not seasoned.	Very Dry.		
Teak, ...	. .	5.40	6.70	1 : 1.24
Birch (American), ...	. .	5.21		
Elm, ...	. .	4.61		
Box, ...	. .	4.36	9.00	1 : 2.06
Ash, ...	3.88	4.18	7.60	1 : 1.82
Mahogany, ...	3.66	3.66	3.60	1 : 1.00
Beech, ...	3.45	4.18	5.00	1 : 1.20
Oak (Dantzig, very dry), ...	. .	3.45		
Baywood, ...	3.36	3.36		
Elder, ...	3.33	4.48		
Sycamore, ...	3.16			
Alder, ...	3.05	3.11		
Pine (Pitch), ...	3.03	3.03		
Deal (white), ...	3.03	3.21		
Fir (spruce) and Crab, ...	2.90	3.04	5.5	1 : 1.80
Oak (English), ...	2.89	4.49	4.5	1 : 1.00
Walnut, ...	2.71	3.23		
Deal (red), ...	2.57	2.94		
Cedar, ...	2.53	2.62		
Pine (red), ...	2.41	3.36		
Pine (yellow, full of turpentine), ...	2.40	2.43		
Hornbeam, ...	2.02	3.25		
Oak (Quebec), ...	1.89	2.67		
Plum, ...	1.63	4.18		
Birch (English), ...	1.47	2.86		
Larch (fallen two months), ...	1.43	2.49		
Poplar, ...	1.39	2.29		
Willow, ...	1.29	2.74		

289. The length of some of the specimens in the column headed Very Dry was but one inch, which, to some extent, may have increased their power of resistance; but no doubt whatever can remain as to the importance of using only dry and well-seasoned timber in construction.

The comparison of the ultimate resistance of timber to compression and tension, respectively, as given in the last column, is very important: it will be seen that in mahogany and English oak the numbers are practically equal, while in boxwood the tensile resistance is double the compression: in the others the excess ranges from $\frac{1}{5}$th to $\frac{4}{5}$ths, the comparison being with the column "Very Dry" in compression.

290. In order to determine the law of the strength of long pillars of timber, the following experiments were undertaken by Mr. E. Hodginson on Dantzic oak and Red deal:—

291. TABLE, *showing the Law of the Strength of long Solid Pillars of Timber, as depending on the Lateral Dimensions and Form of Ends.*

DANTZIC OAK.

	Length.	Side of Square.	Deflections.	Weight of Greatest Resistance.	Remarks.
	Inches.	Inches.	Inches.	lbs. tons.	
I.	60.5	1.75	0.15	3197 = 1.427	Both ends round.
II.	"	"	0.40	6109 = 2.730	One end round, and one flat.
III.	"	"	0.13	9625 = 4.3	Both ends flat.
IV.	48	"	—	9229 = 4.12	" "
V.	30.25	"	—	14,305 = 6.386	" "
VI.	29.75	"	—	13,083 = 5.841	" "
VII.	46.1	1.02	—	1754 = 0.783	" "
VIII.	"	1.50	—	7888 = 3.521	" "

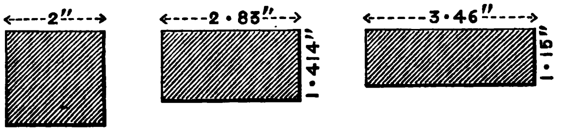
RED DEAL.

	Length.	Side of Square.	Deflections.	Weight of Greatest Resistance.	Remarks.
	Inches	Inches.	Inches.	lbs. tons.	
IX.	58	2	0.24	11,993 = 5.354	Bent in direction of diagonal.
X.	"	1.414 by 2.828	0.25	7681 = 3.43	Sunk by bending in the direction of the smaller side.
XI.	"	1.15 by 3.46	0.42	4349 = 1.941	" "

292. The lines I., II., and III. point out very clearly, by a comparison of the weights of greatest resistance, that the strength of long pillars, with both ends flat, and well bedded, is three times greater than the same piece with its ends rounded; and that the pillar with one end rounded, and

one flat, has double the strength of that in which both are rounded. This truth, which is identical with that derived from like experiments on cast iron, &c., points out the great value of a true and uniform bearing in all pieces of timber which have to act in any framing or structure under compression. The area of transverse section of these pillars being 3.0625 square inches, the resistance in line III. amounts to 1.4 tons per square inch, which is about one-third of the ultimate strength of the short specimen of Dantzic oak in the Table, § 288, showing that the incipient crushing had not commenced with that proportion of strain.

In lines IX., X., XI., the ratio of the sides of the transverse sections are, 1 to 1, 1 to 2, and 1 to 3, respectively, as shown in the woodcut, the area being in all very nearly four square inches.



293. Let us from these experiments endeavour to find the power of the diameter *m*, or of the side of a square prism to which the strength is proportional, the length being constant, and the pillar so long as not to be crushed by the breaking weight. For this purpose the above Table will, in lines VII. and VIII., supply the results of six experiments upon pillars of Dantzic oak, each 46.1 inches long; from which it appears that a pillar 1.02 inch square, sank down, and had its greatest resistance overcome by a mean weight of 1754 lbs., and another 1.50 inch square by 7888 lbs. Now, as the ratio of 1.02 to 1.50 is the same as 1 to 1.47, we shall simplify the calculation by using the latter numbers.

$$\text{Therefore, } 1^m : 1.47^m :: 1754 : 7888$$

and

$$1.47^m = \frac{7888}{1754}$$

Hence, proceeding as in pages 93 and 239, we have

$$m = 3.902;$$

so that the strength is nearly as the fourth power of the side.

With respect to the inverse power of the length, Mr. Hodgkinson remarks—" The great flexibility of such long and slender pieces as would have been necessary to give sufficient variety in their length—and for the length of the shortest to be a sufficient number of times the thickness, that it would not be sensibly crushed by the breaking weight—convinced me that it would be as well to seek for this power hypothetically as to endeavour to obtain it by experiments, which would probably be unsatisfactory." He therefore assumed (after trying some other hypotheses less successfully) the strength of timber to be inversely as the square of the length, so that $\frac{d^4}{l^2}$ represents the law we are seeking, which, as mentioned in page 85, was the simple conclusion of Euler; and if the pillar be of rectangular section, a being the greater side, and b the lesser, the expresison becomes $\frac{ab^3}{l^2}$.

294. We must next proceed to determine, from Table, § 291, the constant for long square pillars of each of these two species of timber, taking the inch as the unit for lateral dimensions, and the foot for lengths, as is most usual in practice. From the average of four experiments on Dantzic oak, line III., we have, proceeding as in § 127, page 95, the value of the constant—that is, the deduced strength of a pillar one foot long, and one inch in the sides, equal to 26085 lbs.; and the average of three experiments, line VIII., gives 22999 lbs. Taking the mean of these two values, we obtain 24542 lbs., or 10.956 tons, which for facility of computation we may call 11 tons; and thus with

$$\text{Dantzic oak, } W = 11 \times \frac{d^4}{l^2}.$$

And with red deal we have in like manner, from the mean of two experiments, in line IX., the pillar being two inches square, the constant equal to 17511 lbs., equal to very nearly 8 tons. Hence with

$$\text{Red deal, } W = 8 \frac{d^4}{l^2},$$

in both the length being taken in feet, and the side of the square or other rectangular dimensions in inches. We may now, therefore, extend the comparison given at the end of § 236; and, taking again the strength of a long flexible pillar of cast iron as 1000, and of wrought iron consequently 1745, that of Dantzic oak will be represented by 108.8, and Red deal by

78.5, all being of the same size. These numbers, except the two last, were all computed from pillars with rounded ends. The several numbers are not, it will be perceived, in the ratio of the numerical constants, being also dependent on the different indices of the dimensions, and the constants for timber having reference to square pillars, which will, therefore, be greater than if they had been derived from circular columns of a diameter equal to the side of the square pillar.

In the pillars of timber with rounded ends it was found necessary to place rounded caps of iron upon the extremities, to prevent them from being crushed. These square pillars were found to bend in a plane passing diagonally through opposite angles, as was also observed in cast iron. In the case of beams, square in the cross section and strained transversely, it will be shown that they are weaker when the diagonal is in the vertical than when the opposite sides are in vertical and horizontal planes.

295. The following experiments on short pillars of Teak wood will serve to prove the latter part of the proposition in § 286, that the resistance of timber to compression, when not long enough to yield by flexure, is proportional to the area of section in action. Twelve cylinders were prepared of ½ inch, 1 inch, and 2 inches in diameter, and of a height double the respective diameters, four of each diameter, the eight last being cut from the same piece of wood. The pressure was exactly in the direction of the axis of the pieces, the ends of which were turned truly perpendicular to that line.

296. TABLE, *showing that the ultimate resistance of short pillars of wood to a compressive force is in proportion to the area of the specimen; ends* FLAT.

Diameter.	Crushing Weights.	Ratio of Weights.	Weight per Sq. Inch.	
Inches.	Lbs.			
½	2439	1.00	12418.5	Average. 12730.5
1	10171	4.17	12950.0	
2	40304	16.53	12823.0	

The areas of the several sets of specimens being as 1, 4, and 16, it appears from the third column that the larger pieces bore relatively rather more than the smaller.

PRACTICAL APPLICATIONS AND EXAMPLES.

297. The utility of timber as applied in practical construction to resist compression, both in permanent and temporary structures, is of the highest character. In Foundations, as used for permanent bearing piles, under massive public buildings, and under the abutments and piers of bridges, it is in many cases indispensable : for instance, when the natural ground is of such character that it cannot be relied upon to sustain the weight intended to be placed upon it without any eventual yielding or settlement—in such sites whole timber balks pointed and shod with iron, are driven vertically into the ground, from 18 to 30 feet deep, according to local circumstances, and the depth at which firm ground may be reached. The lateral distances being from 3 to 4 feet, centre to centre, and in lines at right angles to each other, so that the area of the intended foundation is divided into squares with sides of the above lengths, a pile being at each angle. For about a foot or 18 inches in depth the heads of the piles are, after being driven to the intended depth, surrounded by a bed of concrete, and, being cut off truly level, horizontal sills are laid, and fitted upon the pile heads, to which they are firmly spiked or bolted, and the intervals between the sills being also filled in with concrete, a strong close planking is spiked down upon them, on which the masonry is commenced. In the more massive structures a second series of sills is laid over those immediately on the pile heads, and at right angles to them, and the intervals being filled in with rubble masonry or brickwork, the planking is then spiked down on this second tier. The piles being thus firmly held in their intended position by the lateral compression of the ground along their whole depth, and their heads yet more solidly supported by the surrounding concrete and framing of sills and planking, cannot be looked upon as isolated supports; on the contrary it is impossible for them individually to bend laterally, as long pillars, under their load; nor can the whole of them collectively sway together in any one direction ; for the area of the foundation is always surrounded by a close wall of sheeting piles, driven vertically, which sustains them all within its lines, and also acts to prevent the ground in which they are driven from being disturbed by any external action, such as running water, &c. It is therefore quite impossible to regard the bearing piles as long flexible columns,

but rather in some degree as short specimens, which must yield by actual crushing.

298. At London-bridge, the whole structure weighing about 120,000 tons, rests upon 2092 bearing piles, which gives about 60 tons per pile; and as they were driven at intervals of 4 feet apart, centres, each supports the weight due to 16 square feet of the lowermost course of masonry. In general we may take the area of the pile heads as from 12 to 14 inches square, at a mean about 170 square inches for the area of the head, which would give for the London-bridge 0.353 tons per square inch of pile head, or 50.8 tons per square foot.

299. The bearing piles and platform sills, planking, &c., for the piers of this bridge are shown in plan and section, figs. 202, 203, and were specified to be of Elm, Fir, or Beech timber, not less on an average than 12 inches diameter in the middle, between the head and point, and 20 feet long, properly shod with good wrought-iron shoes, not less than 35 lbs. weight each, and during the driving to have each a good wrought-iron hoop around the head, weighing 30 lbs. All the piles to be driven not less than 18 feet below the under side of the respective platforms, not including the length of the shoes.

The heads having been cut off level and even, sills not less than 12 inches square (of either of the same descriptions of timber) to be laid, and fitted solidly on the pile heads in the transverse direction, and firmly spiked to each pile with jagged wrought-iron spikes 18 inches long and $1\frac{1}{8}$th inch square. At right angles to these, and crossing them over the pile heads, other sills to be laid and spiked down, having the same dimensions, and in the same manner as the first-mentioned. From 9 inches below the pile heads up to the level of the upper sill the spaces to be filled in with concrete brickwork, and good stone masonry grouted in; finally, planking well fitted, close jointed, and 6 inches thick, of Elm, Fir or Beech to be laid in mortar between the timbers, and firmly spiked to the upper sills with spikes 12 inches long and $\frac{3}{4}$th inch square.

300. Sheeting piles to be driven around the entire foundation of the piers, the timber being Fir, not less than 12 inches square, the whole to be well straightened and planed, and driven perfectly close, to be connected together with half timber fir walings, well bolted and secured to the piles, which are to be hooped and shod with wrought-iron shoes not less than 36 lbs. weight each. The above sheet piling to be pitched 18

feet long each pile, and this part to be commenced previous to the main foundation, in order to facilitate getting down to the required depth for the foundation. It will be perceived that no part of the weight of the superstructure rests on the heads of the sheeting piles, avoiding thus a grave error in practical construction, which may be too frequently observed, especially along the front of retaining walls.

301. It is usual in specifying for bearing piles to state that they are to be driven until, with a certain number of blows from a ram of given weight, they shall not sink more than a stated fraction of an inch. Most probably, however, the elder Rennie knew by the experience of some other work the dense and uniform character of the London clay, which, covered with a variable thickness of gravel, underlies every part of the bed of the Thames, and, therefore, may have felt himself secure by only ordering that the piles should be driven to a stated depth. The event, at least, has justified his practice in this particular case, as his noble work stands without the slightest observable settlement in any part.

302. The action of the thick planking, and of the double series of massive sills above described, is evidently to distribute the weight of the superincumbent structure uniformly upon each pilehead, and also to render the resistance of the piles themselves to a uniformly distributed load more equable, each of these essential points being secured by the resistance of the horizontal sills, as beams, to flexure; the superstructure being thus rendered safe from all danger of unsightly and injurious settlements above.

303. The bearing piles under the piers of the Royal Border bridge over the Tweed, the structure of one of the piers of which is shown in figs. 204, 205, are stated by Mr. Bruce in his paper describing its construction, vol. x., Min. Proceedings Inst. Civ. Eng., to carry each about 70 tons. "The standard of intensity which was fixed for the driving of these piles was one inch by 150 blows at the end of the driving; but in two of the piers the driving did not average more than one inch by 20 blows. This, though experience has proved it to be sufficient, is considered too easy driving to be trusted with such a load, if circumstances admit of their being driven harder." The pile engine worked by hand had a ram of 15 cwt., with a fall of 16 feet, and the Nasmyth steam pile-driver had a ram of 30 cwt. falling through 3 feet, and a fixed weight of 42 cwt., that is, of the case or frame, &c., which rested on the pile head during the driving.

The bed of the river consisted of a firm, sound gravel, resting, in some cases, on solid rock. The arches were semicircular, the span being 61 ft. 6 in.; the greatest height above the bed of the river being 126 feet.

304. The bearing piles were chiefly of American Elm, as it was found to stand the hard driving better than Memel, which required to be so often cut off at the head and re-hooped, when driven by Nasmyth's pile-driver, that the delay and expense in driving them was very great when the ground was hard. The cost of driving amounted to 1*s.* 5¼*d.* per foot for the Elm, and 2*s.* 6*d.* for the Memel piles. The shoes with which they were pointed were of cast iron, and weighed 38 lbs. each.

The comparative cost of driving with Nasmyth's engine and the common hand-ram is very much in favour of the former. The hand-ram driving cost 2*s.* per lineal foot, and the steam-piler 1*s.* 0½*d.* in like circumstances—namely, in the bed of the river, and not surrounded by others, or enclosed by the coffer dam. The hand-ram gives one blow in every four minutes, and the steam-piler 240 blows in the same time—or 60 blows per minute.

305. If the total weight of any structure at the lowest course of footings has been ascertained, and the foundation should require to be constructed artificially on bearing piles, we may calculate the number of such bearing piles that must be driven when these two preliminary points have been determined, namely, the greatest load per unit of surface, either the square foot or the square inch, to which the kind of timber should be subjected; and secondly, the most convenient and economical dimensions for the timber of the piles, whether round or square, at the locality of the works; from these two data we compute the load each pile can sustain, and, dividing this number into the given weight of the structure, the quotient will be the number of piles required. It only remains, then, so to distribute them over the area of the foundation that each pile may support its due proportion of the weight, and to mark their position on the plan of the base, figuring their distances from centre to centre.

Thus, suppose the total weight on the base of the river pier of a viaduct amounted to 6650 tons, and if it was deemed prudent to limit the pressure on the heads of the bearing piles, say to 54 tons per square foot, which is equivalent to ⅜th ton per square inch, and also that the dimension of the head found most suitable at the locality was 13 inches square; we have, consequently,

$$13 \times 13 \times \tfrac{3}{8} = 63.375 \text{ tons}$$

as the load for each bearing pile, which divided into the total weight gives

$$6650 \div 63.375 = 105 \text{ as the nearest whole number.}$$

These arranged at 4 feet from centre to centre would give support to the cap sills for a flooring 25 feet by 57; there being 15 piles in each line, and 7 such lines. Experience has pointed out that it is necessary to give the same solidity to the artificial foundation under the cutwaters, though they only extend up to the springing of the arch, as to that part immediately under the pier itself.

If the bearing piles had been of round timber, 15 inches in diameter, then the area of the head will be equal to $(\tfrac{11}{14} \times 15^2 =)$ 176.8 square inches, and $176.8 \times \tfrac{3}{8} = 66.3$ tons per pile, so that a less number at somewhat greater distances apart might be employed in this case.

The following rule is given by Arthur Morin, from Rondelet, namely, that "bearing piles in foundations may be charged with a weight of 30 to 35 kilogrammes per centimetre square," equivalent to about 0.19 to 0.2223 tons per square inch, or from 27.4 to 32 tons per square foot of the pile heads.

306. To drive these very piles into the ground, a weight in the ram of the pile engine of about 15 or 20 cwt., falling freely from 15 to 18 feet, is employed; great care is consequently required in the operation lest the timber should be split, and so destroyed, or the head damaged by the blows it receives, although finally its duty will be to sustain for ages, without any subsidence, a statical load so far out of all proportion to that which forces it down, affording a practical illustration of the great difference between impact and pressure.

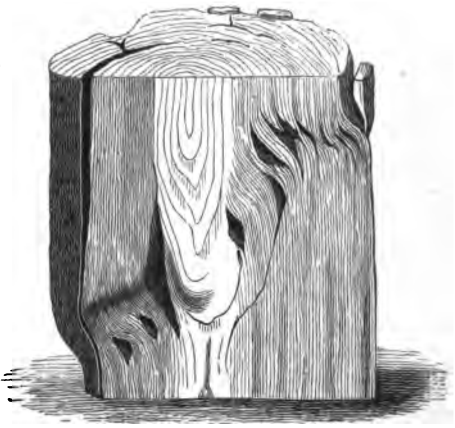
A pile may be trusted for temporary work to sustain about 15 tons when driven, so as not to sink more than $\tfrac{1}{4}$th inch with 15 cwt. weight falling 6 feet.

307. The piers of viaducts of masonry are generally designed so as to have a continually increasing area from the springing of the arch downwards, both by a batter, or outward slope, on every face, and also by bevilled offsets at intervals, and still more by the projection of the courses of footings. If a due attention to the bed bonding of the courses and to

the dimensions of the stones on the outer face has been given, then the uniform distribution of weight on the timber platform will be secured, and its amount per unit of area not greatly augmented with the increase of height; all this care in the design seems at first sight to be reversed below the masonry, for the flooring is supported on the sills, which are about one-fourth of the platform, and these again on the heads of the piles, which terminate with the points of the iron shoe.

But though bearing piles are generally specified to be driven through the loose upper strata about two feet into firm ground, it is not this alone that secures the structure; the whole area being, as already mentioned, enclosed by close sheeting piles outside the line of the platform, it is evident that the ground inside must be greatly compressed by driving such a volume of timber into a confined space, and this increases the friction on the surfaces of the piles, and also counteracts their bending under the load, to which circumstances their efficiency as supports must be chiefly attributed; moreover, the concrete and rubble around the pile heads and under the sills give considerable lateral support to them, and also directly to the planking, and so far relieve the bearing piles.

It is necessary to state that the great expense of timber foundations has been obviated in many recent structures by the use of Portland cement concrete, which sets into one solid mass like a level bed of rock, and cannot settle down even into the loosest ground, when it is properly enclosed in close sheeting piles, as will be mentioned under that heading.



308. The wood engraving illustrates by a vertical section the method of failure of a short pillar of timber. During the raising of one of the tubes of the Britannia bridge the bottom of one of the hydraulic presses by which it was being lifted blew off: and, in the words of Mr. E. Clark, "On the occasion of the

failure of this press, the extremity of the tube fell on to a bed of soft deal planks, piled loosely on each other, about 4 feet high, and about 12 feet by 5 feet in superficies; the tube being about 1½ inches above the planks, and compressing them through a space of about 7 inches, the bulk of the weight of half the tube, or about 1000 tons, was, at the same time, supported by an internal column of elm 14 inches high, and 14 inches square, which continued to support a great part of the weight, although crushed, as in the sketch." The fractured beams and side pillars of the internal cast-iron frame permitted the central pillar, which was uninjured, to transmit to the timber pillar placed immediately under it the enormous weight above mentioned. It will be observed that the crushing of this piece of timber took place in an inclined plane, making about 48° with the horizon, analogous to that of cast-iron, as shown in pages 77 to 82.

309. In the commencement of the railway system, timber viaducts were extensively employed, but experience has proved that their comparatively low first cost, and the admitted facility and rapidity of erection that they allow of, does not compensate for their want of durability. Meantime, the successful introduction of wrought iron has led to their reconstruction generally with girders of malleable iron in preference to the continual expense of maintenance, or of complete renewal, indispensable with the original material.

In these countries, which have long ceased to supply native timber for home consumption, all that is employed for these purposes is imported (to the extent of nearly 3½ million loads per annum), and in which iron, on the other hand, is so abundant and so largely exported, it was not to be expected but that the vastly more durable home material would soon completely supersede the other, especially when, as in the laminated arches of timber, and some other forms of superstructure, the supports were constructed of masonry, easily adapted to bear horizontal wrought-iron girders at the required level. At the present day, when the proper proportions of iron beams are so well understood, and their cost so much reduced, no engineer would be justified in proposing to cross a valley by means of a timber structure, unless some very cogent reasons could be assigned for the preference.

310. The late Mr. Brunel employed timber most extensively in the lines he completed through Cornwall and South Wales, for which he has given some very original and successful designs. That crossing the flat valley of the Tawe,

1760 ft. wide, at Llandore, near Swansea, though not the loftiest, is perhaps the best example, and the local circumstances seem to justify the use of timber in it.

The Admiralty decided that the headway for navigation over the Tawe should be 75 feet above ordinary high-water mark, and the width of the stream required a span of 100 feet in the clear. The low and marshy character of the land on the east side, covered with several feet of peaty soil, and constantly overflowed by the river in floods and at high spring tides, would have formed a bad foundation for an embankment leading up to the Tawe, at the height rendered necessary by the level of the line of rails, as regulated by the headway over the river above mentioned; a very high rate of compensation was also demanded for the land itself.

On the west side, the proximity of the summit level between Swansea and Carmarthen, which it was necessary to attain in a short distance by a rapid gradient, opposed any lowering of the line of rails whatever the means of support adopted, and many buildings had to be cleared away, even for the narrow viaduct employed, rendering the purchase of land for it expensive—much more had the wide base of an embankment to be provided for. The number of openings also required to suit the river, the canal, the tramroads, and railway, and public and private roads, were an argument in favour of some form of continuous viaduct, instead of an embankment pierced at every crossing by a separate arch, with its costly wing walls, &c. The ground was also considerably underworked by coal mines, and the depth to the first coal-measure was only about 200 feet, so that arches of masonry, which are so easily injured and even destroyed by the least subsidence of their piers, would have been unsafe in such a position. Borings carried down to about 30 feet showed the general character of the soil to be blue clay, sand, gravel, and boulder stones.

It may also be stated in favour of using timber in this viaduct, that the site of the works, so near a large port, and over a tidal stream, obviated all the expense of cartage over country roads, which is a considerable item in the cost of such structures when erected over inland or secluded valleys. Such are some of the local circumstances, as given by Mr. Fletcher in his account of this work; and it must be evident that no general rule can be given; the designs and the material employed must, in each case, be left to the discretion of the Engineer, who should be guided in his selection and mode of construction by the circumstances of the locality, and the facility of obtaining different materials, and also by the purposes which the work was intended to serve.

311. The supports, or legs, as they were termed, of the main span, Plate XIV., Fig. 206, acting as piers, consisted of 16 timbers, each 16 inches square, at either abutment, or 32 in all, arranged in groups of four each; the three next the river were vertical in elevation, and bolted together by nine screw bolts 1½″ in diameter, forming a pillar 16 inches by 48, and about 62 feet high; the fourth sloped backwards at the rate of 1 in 10, so that each group had a section 16 inches by 64: all are fitted into cast-iron shoes, resting on oak sills, laid on piles, which are waled on the face, for protection against the river traffic. A sufficient platform for the castings and sills at the base was obtained by driving bearing piles into the ground on the river bank, on which the horizontal sills were bolted. The raking piles, four on each side, gave great stiffness in the longitudinal direction, and transversely this was secured by diagonals between the contiguous sets of legs, and by the batter of 1 in 10 given to two outside ones at each abutment.

The legs supporting the 42 feet spans, Fig. 208, &c., and all smaller openings, were five in number, one under each truss; those supporting the outer trusses have an inclination of 1 in 10, whilst those supporting the centre and intermediate trusses are perpendicular. All these legs are composed of two pieces of timber 12 inches by 14 inches, fastened together by inch bolts, at intervals of 6 feet apart. The timbers composing the two intermediate legs are blocked 9 inches asunder, to allow the passage of the diagonal strut between them. These legs are capped and shod with castings at the top and bottom, and stand on a sill 2 feet 8 inches by 9 inches; no holding down bolts were found necessary for the castings or the sills, either on the masonry or on the pile foundations. The legs are held at the right distance from one another at the top, by short struts, pockets for them being prepared in the castings terminating the legs. Two wrought-iron bolts, one inch diameter, one on each side of the strut, pass through the five caps on the top of the legs, and are tightened up by nuts on the outside of the outer castings.

Transverse rigidity is given by diagonals 9 inches by 9 inches, butting into castings, and passing between the intermediate legs, opened for this purpose.

From the very detailed and clear statements of the author we may deduce the total weight and weight per unit of area at the foot of the legs, taking the timber at 50 cubic feet to the ton.

	Tons.
The half main span—	
Timber,	70.65
Cast iron,	5.50
Wrought do.,	17.85
Permanent way and greatest running load,	110.00
The half 64 feet span—	
Timber, cast and wrought iron,	28.00
Permanent way and greatest running load,	64.00
Weight of each pile, 62 feet long, very nearly 2 tons, and 16 on each bank,	32.00
	328.00 tons.

this divided by 16 gives the total pressure at the foot of each pile at 20.5 tons, which divided by the area $(16^2 =)$ 256, gives about $\frac{1}{12}$th of a ton per square inch, or 11.6 tons per square foot.

In speaking of the trusses the author remarks: "The pine timber in compression was calculated to bear $\frac{1}{6}$th of a ton per square inch; but where there were three trusses abreast in one span, the strain on the centre truss was allowed to exceed this by 50 per cent., which would give $\frac{1}{4}$th ton per square inch."

The 42 feet spans, supported as above mentioned, compressed the feet of the vertical timbers at the rate of about 8 tons per square foot, or $\frac{1}{18}$th of a ton per square inch.

312. The South-Eastern Railway from London to Dover is carried along the coast on a timber viaduct, figs. 210, &c., near the foot of the cliffs, at about high water line, close to its entrance into Dover. The several series of piles, three in each stack or pier, are driven into the firm ground at intervals of 21 feet from centre to centre. If we suppose that trains are passing simultaneously on the up and down line, and that the weight of the engines, permanent way, girders, &c., on each line, amounts to 2 tons per foot forward; it follows, as we have 10 feet longitudinally on each side resting on one stack of piles, that it must support about 80 tons in the total, or 27 tons on each pile. The uniform distribution of the load on each pile is very well effected by the strong and rigid crowntree 12″ × 24″ deep, formed of double balk of whole

timbers bolted together, the uppermost being of greater length, in order to give width for a footway for the workmen attending to the permanent way, &c. The strong waling pieces on each side of the piles, 12 inches deep by 6 wide, bolted through and through, at the level of high water, in effect shorten the piles as pillars, and so obviate their bending; the struts B throw a portion of the weight off the centre pile on the two outermost, and also resist any lateral movements.

313. The timber pier constructed by Mr. Brunel at New Passage, on the east side of the Severn, is shown in transverse section in fig. 212, and in side view in fig. 213. It serves to carry the line of the Bristol and South Wales Junction from high water line to deep water, where the steam ferry-boats can come along side at all states of tide, and from the flatness of the shore is necessarily of great length. The piles are, it will be seen, double balks of timber, about 14 inches square, bolted together, which, as the distance from land increases, are of considerable length, the extreme rise and fall of the tide being nearly 30 feet; accordingly when it was impossible to obtain timber of sufficient length to form them in one piece, two or more breaking joint with each other, as shown at A, were superimposed, a plate of wrought iron being placed between the butt ends of the balks and turned up at each end; the whole secured by a strap of wrought iron bolted on the face of the joint, and fastened by screw bolts through both the jointed and the continuous pile; the double piles are pointed with single cast-iron shoes, and were driven together by a pile engine, with a ram of proportionate weight striking both pieces simultaneously. Waling pieces and diagonals of half balk, bolted through and through, secure and stiffen the piles in the transverse plane.

The section of the girders, which consist of two whole balks bolted together vertically, and short corbelling pieces are shown at B; it will be observed that the double piles are each cut away at the head on the inner face for half their thickness after being driven, forming an excellent shelf for the support of the beams, the weight coming directly on the grain end, the two half piles not cut away staying them up laterally; the outermost pile is scarfed to receive a strong post supporting the hand rail, as shown at D.

The spans, or distances between the several successive stacks of piles, were about 20 feet, and at about every tenth or twelfth opening diagonals of whole timber, intersecting in the middle of the span, were placed, which braced together two contiguous piers or stacks, forming an abutment so as to resist any tendency in the pier to move in the direction of its length—a precaution quite necessary from the exposed position and heavy sea often occurring in this part of the Severn. A strong planking, 6 inches thick, supports the ballast in which the longitudinal sleepers are bedded, in the same manner as on any other part of the line; and it will be observed that the rails are not immediately placed over the line of girders, which has the effect of distributing the passing loads more generally on all the horizontal and vertical supports. The section shown in fig. 212 gives the construction at about one-half the distance from the land to the end.

The whole terminates with a T head containing all the necessary appliances for goods and passengers, and stairs with landings at different depths to suit the state of the tide.

Iron has been successfully applied in structures with the same objects as those in the two last-mentioned works formed of timber, and in localities in a great degree similar, that is, in carrying a line of railway along the head of a tidal bay or estuary within high water line, and also as a jetty or pier to enable the rails to reach sufficiently deep water.

314. Mr. Hawkshaw has carried the line of the Sheffield and Huddersfield Railway over a deep valley on a timber viaduct 1050 feet long, and adopted a principle of construction very simple and efficient in character, resembling a staging. The piers are placed at distances of 16 feet apart on the average, and are founded on a base of rubble masonry, on which are laid horizontal sills; vertical pieces of timber rest on these, and support the girders, which, at such short spans, require no trussing; each pier is well secured transversely by diagonals; and, as the outer piles have all the same rate of slope, the transverse length of the sills which form their bases on the masonry increases in proportion to the depth of the valley, which at the greatest was about 108 feet: and on the face of this outer slope diagonals are bolted, uniting the several supports in the longitudinal direction.

This form of timber viaduct has great resemblance to those which have been erected on some lines in Norway and other countries, having abundant supplies of native timber; with us it has this advantage, that the substitution of a viaduct in arches of masonry, or with wrought iron girders, can easily be effected without interruption to the traffic.

315. The duration of the planking under the ballast may be taken at about twelve or fourteen years; and that of the main

timbers at nearly double the above period. Much, however, depends on the seasoning of the wood when first put in.

316. **Table,** *giving a Statement which will serve to point out the comparative Cost of Viaducts constructed of different Materials:—*

Name, and Material used in Construction.	Length.	Extreme Height.	Cost per Foot forward. £.	s.	d.
IRON.					
Crumlin, ...	1500	200	26	0	0
Deepdale,.	740	160	27	7	0
MASONRY.					
Lockwood , ...	1428	136	24	8	0
Lugar, ...	752	162	42	0	0
BRICK.					
Chapple, ...	1130	60	18	10	0
Hownes Gill (if for double line), ...	730	162	28	7	0
TIMBER.					
Landore, ...	1760	88	16	6	0
Denby Dale, ...	1070	108	14	0	0

So that, if the very varied circumstances of locality, &c., do admit of comparison, the cost of bridges of timber seems to be about 60 per cent. that of the other materials.

DIRECT RESISTANCE OF STONE, BRICK, AND CEMENT, TO A COMPRESSING FORCE.

317. **Stone and Brick.**—*The direct resistance of stone to a compressive force varies from about* 11 *tons to half a ton per square inch, according to the formation from which it has been taken.*

The experimental proofs have been derived from a number of different authors, using very different apparatus, and were generally undertaken with a view to obtain data for the details of some particular work, at the site of which they were carried on.

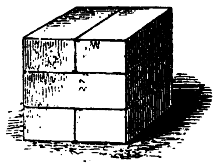
318. TABLE, *showing the ultimate Resistance of various Kinds of Stone and Brick to a Force of Compression.*

Designation of the Stone.	Lbs. per Square Inch.	Tons per Square Foot.	Column to crush, Feet.	Sp. G.	Weight lbs. per Cub. Ft.	Cubic Feet to a Ton.
Mr. Mallet.						
QUARTZ.						
Across Lamination, . . . lbs. 25500						
Parallel to, . . 14000						
Average, ...	19750	1270	17145	2.6545	166	13.5
SLATE.						
Across Lamination, . . . 19500						
Parallel to, . . 18000						
Average, ...	18750	1205	15665	2.7545	172	13
Mr. Fairbairn.						
GRAUWACKE, .	16893	1086	14064	2.766	172.9	12.95
GRANITE, . .	13124	842	11325	2.66	166.5	13.45
SANDSTONES,						
Across Lamination, . . . 10621						
Parallel to, . . 9187						
Average, ...	9904	646	19477	2.414	152.7	14.67
Mr. E. Clark.						
LIMESTONE, .	7339	471	6433	2.6545	166	13.5
SANDSTONE,						
Very dry, . . 2043						
Damp, . . . 1285						
Very wet, . . 1085						
Set in cement 6 inch cube, } 3925						
Average, ...	2185	134	2351	2.118	131	17
Mr. G. B. Bruce.						
SANDSTONE,						
From near Berwick and Alnwick. Strongest, 362 tons, weakest, 67 tons per square foot,	3034	195				
Mr. J. R. Walker.						
FIRE BRICK.						
Stourbridge, . . .	1501	96	1728	2.00	124	18
BRICK.						
Blue, Staffordshire, .	1389	89	1513	2.118	131.5	17
Brown do.	1120	71.8	1213	2.131	132.7	16.9
Highest, 115 tons, lowest, 53 tons per sq. ft.						
Mr. E. Clark.						
BRICKWORK, both as set in cement and between deal boards,	521	35.5	583.7	2.057	128	17.5

319. The experiments on quartz and slate by Mr. Mallet form part of his interesting investigation on the transmission of earthquake waves; they were made with great care at the Woolwich testing machine, and may be regarded as very accurate. It would, however, add greatly to the value of all experiments on compression if the cubical form was altogether laid aside for one in which the height was at least one and a-half times the side of the base. The cube may give too high a result if the plane of least resistance should make an angle greater than 45° with the horizon. Those by Mr. Fairbairn, also, though on small pieces, had the advantage of E. Hodgkinson's skill and experience, and were, for the most part, of a height one and a-half times the base.

Mr. E. Clark, in these as in all his other experiments, was remarkable for employing actual direct vertical weight without the aid of a multiplying lever, and his results are therefore highly trustworthy. The dimensions of his specimen pieces were also much greater than usually employed; the limestone was in cubes of 3 inches in the side, and the sandstone of the same size, one however, being a 6-inch cube; that 9¼-inch in the side resisted the greatest power of the apparatus.

The brickwork was built into 9-inch cubes, weighing on the average 125.6 lbs. per cubic foot, The best quality and highest resistance was 612.7 lbs. per square inch; the lowest, set between deal boards, was 417 lbs. per square inch, which is only ⅔rds. of the former; these bricks were manufactured on the site of the works at the Britannia Bridge, and seem to have been of a quality much below those to be obtained from well-established brickworks; thus single bricks of the following places bore as under:—



Britannia,	Birkenhead,	Buckley Mountain,
1022 lbs.	1775 lbs.	2130.3 lbs.

per square inch, all having been bedded in cement; with these weights they were completely crushed to powder. It would seem also that single bricks give higher results than the same when built together with cement.

All the limestones were obtained from the quarries at Penmon, on the north side of Anglesea; they formed perpendicular cracks and splinters a considerable time before they were finally crushed. The sandstones, which were from Runcorn on the Mersey, all gave way suddenly, and without any previous cracking or warning. After fracture the upper portion generally retained the form of an inverted square pyramid, about 2½ inches high, and very symmetrical, the sides bulging away in pieces all round. It was absolutely necessary to ascertain the ultimate resistance of these three building materials which were so severely strained in the towers, both permanently, and especially during the raising of the tubes.

320. The Britannia Tower is 221 feet 3 inches high, with a batter of 1 in 36 on all sides, the base being 60 feet by 50 feet 5 inches. The foundation stones are placed on a stepped bed blasted in the rock, which consists of chlorite schist. The courses vary in height from 3 feet 3 inches to 1 foot 8 inches: none of the courses beneath the tube are less than 2 feet.

The tower is not solid; two internal rectangular wells extending from the basement to the level of the tubes.

This magnificent tower contains the following quantities and weights of material:—

	Cube feet.	Tons.
Anglesea limestone, ...	151158	
Runcorn sandstone, . . .	127001	
Brickwork, ...	68411	
	346570 =	24700
Cast iron bed plates and beams, . . .		479
Two tubes, ...		4000
Junctions and castings, ...		380
Total in tons, .		29559

The area of this tower at the base is 1847 square feet, consequently the pressure on each square foot amounts to 16 tons, or $\frac{1}{30}$th of the ultimate resistance of the solid limestone. A simple vertical prism or uniform column of this limestone one foot in the base and 216 feet high, that is 5 feet less than that of the tower, would press on its foundation with 16 tons; this will serve to show the skilful distribution of the stone in this tower, which, though 221 feet high and carrying the additional load of the tubes, weighing 4000 tons, has only that pressure of 16 tons per square foot on the base. The internal face of the walls of the rectangular wells were all vertical, the exterior having the batter of 1 inch in 3 feet; this reduces the thickness from 10 feet 6 inches at the base to 8 feet at the level of the bottom of the tubes; above that they also diminished in thickness and the campanile at top was open.

"In raising the tube, the vertical pressure on the masonry of the abutments amounted to 500 tons upon 10 square feet, or 50 tons per foot superficial, exclusive of joints, being about one-eighth of its crushing weight in the experiments."

"The side towers, which are 18 feet less in height than the central one above mentioned, weighed, with 382 tons of cast iron, about 18,000 tons; this, together with 2500 tons, the weight of two tubes, gives a pressure of 13 tons per superficial foot on the lower courses of the masonry."

321. In ecclesiastical structures, both ancient and modern, we have some instances of very great pressures, especially at the intersection of the nave and transepts, where four piers of masonry have to sustain a tower or spire, when built over that point of the cross, or the weight of a dome, as in some of the more recent cathedrals.

322. The following instances are taken from a statement given in Gwilt's Encyclopædia of Architecture (*Ed.*, 1867).

The piers bearing the domes of the several buildings named are computed to bear the weight per square foot at the base placed after each:—

	Tons per Square Foot.
Piers of the Tower of the Church of St. Mary, ...	27
Pantheon at Paris, ...	26.934
St. Paolo, near Rome, ...	18.123
St. Paul's, London, ...	17.705
St. Peter's, at Rome, ...	14.964
Hospital of the Invalides, at Paris, .	13.598

It is not possible to add to this statement the proportion which each of these numbers bears to the ultimate resistance, as we do not possess either the name of the particular stone used in each structure, or any experiments on their several ultimate powers of resistance, except in the case of St. Paul's, which was built of the oolite, from Portland. If we take the general average of this stone from the experiments of the Institute of British Architects, we find that the brown bed had an ultimate resistance of 1.68 tons per square inch, which is equal to 241.92 tons per square foot, or 13.7 times that given in the above statement as to that cathedral.

323. The chapter houses attached to ancient Gothic cathedrals were of a polygonal form in plan, with either eight or

ten sides, and vaulted over with a stone roof, supported commonly on the exterior walls, and on a single central pillar, the diameter in the interior being from 30 to 40 feet. That at Elgin in Scotland is stated to have withstood for centuries a pressure of 40.000 lbs. per square foot, which is very nearly 18 tons. The stone is probably a hard sandstone.

324. In the grain warehouses built at Plymouth by Sir John Rennie, the bases of the masonry piers, consisting of rubble, in courses with good mortar, the material being the hard limestone of the district, are computed to sustain a weight of 22 tons per square foot; they were 4 feet square, and had through courses at intervals of 6 feet.

325. In **Brickwork**, it is probable that the examples of the highest direct pressure will be found in the lofty chimneys of chemical works and manufactories.

The total height of the great chimney at Edinburgh is 341 feet 6 inches; the bed of the foundation was a hard clay shale; the masonry, being 40 feet 6 inches square at the bottom course, had an area at the base of 1640 square feet, and the total weight being 4000 tons, the pressure amounted to nearly 2½ tons per square foot. The total height above mentioned is divided, so that 77 feet 6 inches are allotted to the foundation and square pedestal of stone, and 264 to the brickwork of the shaft proper, the thickness of which was diminished towards the top by five successive steps. Commencing at the top, we have the following dimensions:—

Successive Depths from the Summit.	Thickness, Inches.
83 ft.	15
58	20
48	25
40	30
35	35
264 feet,	

the internal diameter at the top being 11 feet 4 inches, and at the lower part 20 feet. In calculating the weight and pressure of each division, it was found on the first not to exceed 4½ tons per square foot; in the middle it had increased to 7 tons, and at the base to 8 tons per square foot. The strength of ordinary brick being estimated at 20 or 30 tons per square foot, all anxiety was done away with; but a brick formed of a compound of fire-clay and ironstone from the neighbourhood being found to bear a far greater weight than this, it was adopted.

326. Other considerations in addition to those of resistance to pressure must, in these structures, be satisfied—namely, their power to resist oversetting by the force of the wind, and this not at the base alone, but also at different points in the height. Assuming any horizontal section, we have for its stability, omitting the cohesion of the cementing material, the product of its weight into the radius of the base, and, taking the pressure of the wind as having only $\frac{1}{3}^{\text{rd}}$ of the effect on a cylinder to that produced by the same on a plane equal to the vertical section through the axis, its moment will be found by multiplying the total force so computed into half the depth from the top to the assumed section. Now the weight, down to 83 feet from the summit, is 270 tons; and this, multiplied into the radius of its base, was four times greater than the product of the utmost known pressure of the wind into $\frac{1}{3}^{\text{rd}}$ of the section of the shaft for the upper 83 feet, into half that height. At other points the stability was found to be rather less than this, being equal to $3\frac{3}{4}^{\text{ths}}$ at the base.

The chimney at St. Rollox, near Glasgow, was 455 feet high, tapering upwards from 41 feet diameter at the base to 13 feet at the summit.

327. The following additional examples have been calculated by Mr. C. Cowper :—The chimney at Adkin's soap works, near Birmingham, is 312 feet high, the pressure on the base was 6 tons per square foot, and on the foundation below the footings $1\frac{1}{2}$ tons per square foot. It has been much reduced in height, not, however, owing to any defect in the base, but rather to the corrosion of the brick at the top, owing to the large quantity of muriatic acid which was constantly passing up it. The chimney of the Patent Lap-welded Tube Works, near Birmingham, is 145 feet high, and the pressure, owing to six large flues being cut through it, is $8\frac{1}{2}$ tons per square foot of the hexagonal base. A glasshouse cone, 75 feet high, with large arches in the four sides, had 4 tons per square foot on the piers; which is, perhaps, as much as should be allowed when the brickwork is exposed to great heat.

328. From all that has been now given, we may deduce the rule that masonry, of any material, should not be loaded permanently above $\frac{1}{12}^{\text{th}}$ of its ultimate resistance to compression. It has been well remarked that the weakest stone, and even inferior brick, are seldom found really crushed in buildings. But in the actual practice of construction, we

must remember that structures are liable to various occasional and accidental pressures, and to others of a permanent kind, resulting from settlement, and other causes, of which no previous account can be taken, for which allowance must, nevertheless, be made. The pressure on the beds of each course may not be uniformly distributed; and if the resultant should approach the external surface, some flushing, as it is called, or vertical cracks and splinters, begin to appear, especially if the beds of mortar have been too thin, and the stone of one course has come into actual contact with that next to it.

However well the beds of stone, even of Aisler work, may be dressed, the contact, if there be no cementing material between them, takes place in some few small portions or points only, at which, under severe pressure, crushing must begin; and, if these should be near the outer edge, vertical cracks or splinters chipping off, according to the nature of the stone, soon make their appearance. In fact, the true object and utility of a bed of mortar must be to take complete impressions or seals of the two contiguous surfaces of the stone, and effectually separating them, though by a very thin bed, transmit the pressure from one to the other at every point of the surface, and obviate the danger above-mentioned.

329. Portland Cement, after a rather unpromising commencement, has at length obtained a very high degree of favour with Engineers and Architects, both on the Continent and in Great Britain, especially in works exposed to water.

Compared with Roman cement, it has the advantages, 1st, of being adapted for use, both as an hydraulic mortar, when mixed with sand in any proportion desired, and 2ndly, for concrete also, either in foundations or as backing to front work, and even as grout. It is also claimed for it, 3rdly, that it sets harder than the rival cements now nearly supplanted by it; but it has not the property of quick setting, as compared with the natural cements, Pozzolano, Tarras, &c., or the artificial manufactures called Roman, Orchard, &c., cement.

For a length of time the manufacturers had a difficulty in preparing it for builders, to be employed in mouldings, and as external plaistering; but this, to a great extent, is now overcome. It is sold in the form of a very fine powder, and derives its name from its resemblance in colour to the oolite stone from Portland Island, the colour, when of good quality, being a light warm pearl grey.

The process of manufacture may best be described in the words of Mr. G. F. White :—

"Chalk and clay, mixed in certain definite proportions, are carefully ground together in water, and the mixture is then run off into backs or reservoirs. After attaining a specific consistency, it is artificially dried, and is then removed to the kiln for calcination, which operation is carried to a much greater extent than that needed for lime and Roman cements.

"It is next delivered to the mill for grinding, and, after being sifted, is packed for use, the important points of uniformity in colour and in time of setting having been previously ascertained."

This simple process requires, however, great practical skill and unremitting attention; the selection of its two well-known and simple ingredients demands also much care and judgment.

It is chiefly manufactured on the banks of the Thames and Medway, the clay being obtained from the creeks and bays between Sheerness and Maidstone, and the chalk from the contiguous quarries at its outcrop, in the immediate neighbourhood. The clay should be as free from sand as possible, and the proportion in which it is used depends on the quality of the chalk with which it is to be incorporated. In the white chalk districts the clay forms 25 to 30 per cent. of the whole bulk, the chalk being, consequently, 75 to 60 per cent. In the grey chalk districts the clay varies from 16 to 20 per cent., and the chalk consequently from 84 to 80 per cent.

330. The Roman cement, called also Parker's cement from its discoverer, who brought it into use in 1796, is manufactured from nodules dredged up from the clay at the Isle of Sheppy, which is calcined, broken into small pieces, and ground into a fine powder. It probably derives its name from its red colour, which somewhat resembles the volcanic ash imported from Civita Vecchia in the Roman States—a material which it has now, in a great degree, superseded. The Tarras, or Trass, so much used in Holland, is a volcanic ash, shipped generally at Andernach, on the Rhine, where that river intersects the range of extinct volcanoes of Central Europe.

Ever since the time of Smeaton, who pointed out, when investigating the question as to the best description of lime to be used in the construction of Eddystone Lighthouse, that hydraulic limes contained a natural mixture of clay in the stone from which they were derived, it has been diligently sought by Vicat, Paisley, and others, to manufacture an artificial water cement. The final success which has attended

Portland cement is due to perfecting sundry points of detail, such as the degree of calcination, &c., and not to any improved theory on the subject.

331. The tests to which it is generally subjected are three:—1, resistance to a tensile force; 2, specific gravity; and 3, what is called the water test. For the first, a specimen or briquette is formed in a carefully constructed mould, which gives the shape shown in elevation and end view in fig. 219. The dimensions of the narrow central part are always 1½ inch by 1½, which gives a transverse section of 2.25 square inches; the specimens, when being tested, are held vertically in the clips, fig. 220, applied as in fig. 221, which is then placed under the short arm of a steel yard balance, and broken.

The specific gravity is taken by ascertaining the weight in pounds of the striked bushel. We have here an example of the confused, irregular, and unscientific English weights and measures. The standard of capacity is the imperial gallon, containing 10 lbs. weight of water, and the bushel is 8 gallons, or 80 lbs. of water, so that it is equal to 1.2841 cubic feet. Approximately, we have the cubic foot to the bushel nearly as the area of a circle to the square of its diameter, or 11 to 14; and, therefore, 21 bushels are nearly equal to one cubic yard. In general every tenth sack or cask should be taken at random, the weight per bushel ascertained, and sufficient of the dry powder reserved to make a few specimens for the trial of its tensile strength.

The water test is very useful when no means are at hand to apply the two former, and consists in gauging a small quantity of the dry powder with sufficient water, and immediately immersing it in water, when, if the sharper edges crack or break away after a short time, the cement, of which it was a sample, must be set aside, either as being too hot and fresh for use, or as being inferior in quality.

The weight of good Portland cement ranges from 100 to 130 lbs. per bushel, which is equivalent to from 80 to 102 lbs. per cubic foot; the lighter kinds, which are less highly calcined, setting much more rapidly than the heavier; but being far weaker both neat and when mixed with similar proportions of sand.

The bulk that the purchaser receives will, of course, always be inversely as the specified weight per bushel,

Thus 106⅔rds lbs. per bushel gives 1 cubic yard to the ton, and 130 lbs. per bushel only 17.23 cubic feet to the ton.

332. The test specified by Mr. Bazalgette, at the commencement of the Main Drainage of London, was, a tensile resistance of 400 lbs. on an area 1½ × 1½ inches, equal to 2.25 square inches, after seven days' immersion in water, and a specific gravity such that a bushel should weigh 110 lbs.

This was altered in the second specification, as far as tensile strength is concerned, to 500 lbs. on the 2¼th square inches, the weight remaining the same.

The clause, as last amended, is as follows:—"The Portland cement shall be of the very best quality, ground extremely fine, weighing not less than 112 lbs. to the striked bushel, and capable of maintaining a breaking weight of 250 lbs. per square inch, seven days after being made in a brass mould, and immersed in water during the interval of seven days. The contractor shall, at all times, keep in store upon the works a supply of cement equal to at least fourteen days' requirements; and, with each delivery of cement, shall send to the Clerk of Works a memorandum of the number of bushels sent in, and the name of the manufacturer."

The weight of 250 lbs. per square inch is equivalent to 562.5 on each specimen 2¼ inches square.

The specification of the French engineers, who imported this cement in large quantities from England, for blocks of beton in breakwaters prior to its adoption here, is, when changed into our measures—

Specific weight, . . .	103	lbs. per bushel.
Tensile strength on 2¼ square inches when neat,	140	lbs. in 2 days.
,, ,,	280	lbs. in 5 days.
,, ,,	530	lbs. in 30 days.

When mixed—cement, 1 part; sand, 2 parts, ...	140	lbs. in 5 days.
,, ,,	280	lbs. in 30 days.

The average results of the tests of the Metropolitan Board of Works up to 1865 have been, that the cement weighed 114.15 lbs. per bushel, and bore a tensile strain of 600.8 lbs.; equal to 270 lbs. per square inch; thus it has been above proof in both these respects, so that all objections against their standard, as being too high, would seem to be not well founded. This average weight is equal to 19.62 cubic feet to a ton.

If we desire to turn the resistance in lbs. on the area of 2¼th square inches into lbs. per square inch, multiply by 4 and divide by 9; and if into lbs. per square foot, multiply by 64; or, if into tons per square foot, divide by 35.

333. Objections have been raised against the mode of ascertaining the specific weight. Mr. Bramwell states that "A quantity of cement was poured into a bushel measure by a man accustomed to the work, and struck off level. It then weighed 107 lbs. Another portion of the same cement was, secondly, poured slowly out of the sack down an inclined board into the bushel measure, and it then only weighed 97 lbs.; it was then shaken down in the measure, and the weight got up to 132 lbs." By a method analogous to that by which the specific gravity of gunpowder is obtained, Mr. Bramwell ascertained that the specific gravity of the above cement was 3.11; so that if it could be obtained in a state of perfect solidity it would weigh 249 lbs. (= 3.11 × 80), and not 107 lbs., as put into the bushel measure in the manner above mentioned. It is evident, therefore, that ascertaining the weight of this material, or, indeed, of any granulated substance, by pouring it into a measure, is liable to extremely varied results, even with the greatest care.

"The present method of ascertaining the relative weight seems to act as a premium for coarse grinding; for the same cement as above, being sifted through a sieve having 900 holes to the square inch, gave, for that which would not pass through 144 lbs. per bushel when shaken down; and for the finer portion, which did pass through the sieve, 130 lbs.; the treatment being, in each case, the same as that which gave the 132 lbs. per bushel above mentioned."

As to the test by tensile strain, it has been remarked, as an objection to this, that in the same cement this varies with the quantity of water used in gauging it: about one-fourth of the weight of the cement is the least that can be used. Two samples were prepared of the same cement, one mixed with $\frac{1}{3}^{rd}$, the other with $\frac{1}{2}$ its weight of water; the former broke with 244 lbs., the latter with 90 lbs., at the end of ten days in both cases.

The same workmen are, however, generally employed at the testing operations on large works; and they will acquire, in time, an identical method of performing their duties, so that the comparison between different deliveries will stand good, and point out any falling off, or the contrary, in the quality supplied, though not absolutely correct.

Architects and engineers who may require but small quantities on works where the expense of a testing machine (about £50) would not be justified, must at all times feel anxious as to the quality they receive, so much bad cement being always in the market. It is usual, under such circumstances, to specify that the cement shall stand two winters without any sign of failure; and this stipulation has been successfully enforced in cases where bad material had been at first employed.

334. The tensile strength of this cement increases with age, both when used neat and when mixed with various proportions of sand. The diagram, fig. 222, taken from Mr. Grant's paper on Portland cement, exhibits this important result. The horizontal scale represents Time, at one inch to a month; and the vertical scale represents Weight, at half an inch to 100 lbs.

Each vertical ordinate shows the number of pounds with which the cement broke under the tensile force, on a sectional area of 2¼ square inches, every numbered result being an average of ten tests; the specific weight being 112 lbs. per bushel. The specimens were kept in water during the whole time between being moulded and tested. The strength of neat Roman cement, the weight of which is generally 80 lbs. per bushel, is shown in the darker line, where written.

The Roman cement has, from fig. 222, evidently the same property of increase of strength with age that the Portland exhibits; but its relative inferiority remains unaltered, having, at the end of the year, but little more than *one-third* of the ultimate tensile resistance of the other.

The importance of using clean sharp sand in making mortar, and of washing the inferior qualities before mixing, has long been recognised; and the diagram, fig. 222, in the second, third, and fourth curved lines, confirms this in the case of cement. Mr. Grant has found that the increase of tensile strength with washed sand, compared with the same when unwashed, ranges from 30 to 35 per cent.

The diagram, fig. 223, exhibits the relation of the strength to age for a still longer period, both when neat and gauged with one of sand to one of cement. The weight of the cement was, in this diagram, 123 lbs. per bushel; the sectional area and circumstances of immersion being the same as in the series of experiments represented in the other diagram fig. 222. It is intended to continue these experiments for a total period of ten years. At the latter end of the third year the strength seems rather to have passed the maximum point.

335. Looking along the numbers in the three lines in which the proportion was one cement to one sand, it will be perceived that the rate of increase of strength rapidly gains, in proportion, upon that of the neat cement. Thus, loamy pit sand, in which the increase was only 21.8 per cent. at the end of a week, and 45.5 per cent. at the end of a month, had, at the end of a year, increased to 65.1 per cent. of the strength of the neat cement. With the use of clean pit sand, the corresponding line for equal proportions shows that the gain is much greater, increasing from 34.2 per cent. at the end of a week to 74 per cent. at the end of a year.

336. TABLE, *showing the increase of the Tensile Strength of Portland Cement as the Weight per Bushel is increased.*

Average Weight per Bushel.	Tensile Resistance on 2·25 sqr. inches.	Average Weight per Bushel.	Tensile Resistance on 2·25 sqr. inches.	Average Weight per Bushel.	Tensile Resistance on 2·25 sqr. inches.
lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
106	472.6	114	699.7	123	673.6
107	592.3	115	705.5	124	819.9
108	650.1	116	768.3	125	816.2
109	646.6	117	718.4	126	657.2
110	708.3	118	644.1	127	864.6
111	693.8	119	777.9	128	916.6
112	687.5	120	732.3	129	920.2
113	701.5	121	705.6	130	913.9
		122	716.6		

337. The above results prove in general, that "weight, when legitimately obtained by burning and good manufacture, is strength," though some exceptions do occur, as, for instance, with 118, 123, and 126 lbs. per bushel; each result, however, is deduced from a large number of experiments on a total of 327136 bushels. It thus appears that adding one-fourth to the weight, that is raising 106 lbs. to 130 lbs., nearly doubled the tensile strength, which was 472.5 and 913.9. lbs. repectively.

It is important to know that the lighter cements set more rapidly than the heavier, and are consequently at times employed in practice when that property is of value, even though ultimately not so strong as the heavier; thus, in the beds and joints of the front work of masonry exposed to running water soon after being built, it would be essential to employ some lighter Portland cement for a few inches in from the face, or even another description of cement, as Orchard or Medina. Roman cement has been very generally used to point the beds and joints of masonry set in hydraulic mortar, when the work was exposed to a rising tide.

If the strength of a light cement be compared with that of one weighing much more, both being prepared at the same time, a very misleading result may be obtained if the comparison is made too soon; for the light cement very early arrives at its greatest strength, passing that of the other, which more slowly arrives at its highest point, and in the end greatly exceeds the lighter specimen.

338. The ultimate resistance of Portland cement, and its compounds with sand, under a compressive force, has been investigated by Mr. Grant, chiefly in relation to that of the various descriptions of bricks and stone employed on the works of the Metropolitan sewerage, and in pieces of the same shape and size as the bricks, namely, 9″ × 4.25 × 2.75″, all being laid on the flat, so that the area compressed was about 9 × 4.25. The results are, therefore, limited to a comparison between these materials when of that particular form, and when laid on the larger face so that the height was less than either of the other dimensions. Some few, however, were in the form of cubes and prisms.

Taking the average of the good stock bricks, and excluding the results of those of the highest quality formed of fire clay and of gault, we find that the resistance, when crushing first commenced, amounted to nearly 6 tons; and, on the other hand, that of a mixture of one of Portland cement and five of sand, age 3 to 6 months, to about 12 tons, or twice as strong; so that the cement brick would have borne a much larger proportion of sand, before its power of resistance to compression had been thereby reduced below that of the stock bricks.

These experiments have led to the expectation that bricks formed of this cement and sand might be found a cheaper building material than those formed of burnt clay, but nothing has yet been done in that direction.

Concrete, with proportions of 1 cement to 10 gravel, crushes at 15 to 16 tons per square foot, age 3 months, and costs from 10 to 14 shillings per cube yard.

339. TABLE *showing the Resistance to Compression of Blocks of various Substances. The Crushing Force was taken at the first Symptoms of Failure..*

MATERIALS AND PROPORTIONS.	RESISTANCE TO COMPRESSION.		
	Lbs. per sqr. inch.		Tons per sq. foot.
PORTLAND CEMENT CONCRETE.			
Cement, 1; Thames ballast, 6. Size, 12 in. cube; Age, 9 months.	838	=	54
BLUE LIAS LIME CONCRETE.			
Lime, 1; Thames ballast, 6; two specimens 12 in. cube, and one 12 by 7½, and 6 inches in height. Age, 9 months.	178	=	11.5
BRICKWORK.			
Built with 1 Portland cement and 1 sand; Height, 9 inches, and area of base, 166.5 sqr. inches on the average. Age, 1 year.	258	=	16.6
PORTLAND CEMENT BLOCKS.			
Bearing area in all, 144 sqr. inches; Height, from 10½ to 12 inches. Age, 1 year.			
Neat Portland Cement, ...	Not crushed with 100 tons.		
Cement, 1 .... Sand, 1............	1478	=	95
" 1 .... " 2............	1167	=	75
" 1 .... " 3............	731	=	47
" 1 .... " 4............	404	=	26
" 1 .... " 5............	358	=	23

The result above given for Portland cement concrete, namely, 54 tons per square foot, is the average of five experiments; a sixth is given, but it would seem to have been anomalous, as the resistance amounted only to 17 tons. The average of all six would be 724 lbs. per square inch, or 46.6 tons per square foot.

Comparing the resistance of the blocks made with cement and those of blue lias lime with the same proportions of ballast, it will be perceived, from the two first experiments, that the former has nearly five times higher resistance to compression, and therefore it has very generally superseded the other for the foundations of heavy structures.

340. TABLE, *founded on 178 Experiments, showing the Resistance of Portland Cement, &c. &c. to Compression.*

BRICK-SHAPED SPECIMENS, AVERAGE 9 × 4.25 × 2.75. SURFACE UNDER PRESSURE, 9 × 4.25 = 38.25 squares.	AGE OF SPECIMENS.		
	Three Months.	Six Months.	Nine Months.
	Tons. Tons.	Tons. Tons.	Tons. Tons.
Neat Portland Cement, ...	42 .. 65	58 .. 92	37 .. 102
1 vol. of Cement, 1 vol. of Sand, ...	29 .. 43	33 .. 59	64 .. 78
1 " " 2 " "	26 .. 34	25 .. 47	51 .. 62
1 " " 3 " "	20 .. 24	19 .. 37	33 .. 41
1 " " 4 " "	21 .. 23	10 .. 31	25 .. 38
1 " " 5 " "	10 .. 16	12 .. 26	19 .. 29

The first columns show the resistances at the earliest signs of giving way; the second, when the specimens were finally crushed.

The following Memoranda are interesting and useful:—

341. TABLE, *showing the Averages of two Series of Experiments as to the Quantities of Portland Cement, Sand, and Water, used in making one Cubic Yard of Compo or Cement Mortar. The Quantities are given in Bushels.*

RATIOS OF CEMENT AND SAND.	1 to 1	1 to 2	1 to 3	1 to 4	1 to 5
	Bushels.	Bushels.	Bushels.	Bushels.	Bushels.
Cement, ...	12¾	8⅜	6⅜	5⅛	4.2
Sand, ...	12¾	16¾	19⅛	20½	20.9
Water, ...	6½	5	4⅔	5⅓	5.3
Totals, ...	32	30⅛	30⅙	31	30.4

The average being 30.74 bushels to make one cubic yard, or 21.2 bushels of mortar; so that the shrinkage is nearly one-third. Multiplying the bushels of water by 8, we obtain the number of gallons of water required in each case.

342. The weight of a cubic yard of concrete formed of 1 cement to 8 Thames ballast, wet as put into the work, is 30½ cwt., or nearly one ton and a-half per cubic yard, and, consequently, 18 cubic feet are equal to one ton.

In the form of concrete the shrinkage of the cement and gravel, omitting the volume of water employed, is about 25 per cent. Thus, the large blocks used at Dover pier, containing 112 cubic feet when moulded, required about 13 cubic feet (i. e. 10.2 bushels) of dry cement, and 136 cubic feet of dry shingle, giving a total of 149 cubic feet, from which if we take 37 cubic feet, or one-fourth, the remainder will be equal to the finished block. The proportion of cement to shingle above mentioned is about one to eleven, which experience proved to be sufficient at that work. The shingle was taken from the beach, and is described as "consisting of large and small clean shingle and coarse sand, in nearly equal proportions."

If, therefore, the drawings of any work showed that a certain volume of concrete would be required, by adding one-third we obtain the total quantity of materials to be provided, and one-twelfth of this will be equal to the cement required, if the ratio be one to eleven. Thus, if 3600 cubic yards of concrete are shown in any design, it will require 4800 cubic yards of materials, that is, 4400 cubic yards of gravel or shingle, and 400 of cement, or nearly 8480 bushels.

The following experiment nearly confirms the above. Gravel and Lime, mixed in the proportions of eight to one, namely, 11664 measures of gravel, and 1458 of lime (a total of 13122 measures), made 10368 of concrete, showing a loss of bulk equal to 2754 measures (13122 − 10368), which gives 21 per cent.

On the gravel alone the volume is diminished rather more than 11 per cent., namely, 11664 − 10368 = 1296. A contract in which 2000 cubic yards of concrete were employed, showed that 2200 cubic yards of gravel were required.

343. Statement of the quantities used in making brickwork blocks in compo, composed of Portland cement and river sand in equal proportions, each block being a 3 feet cube, or one cubic yard.

The cement weighed 112 lbs. per bushel, and each block contained 384 bricks, which were about 9 × 4.17 × 2.75 inches, and averaged about 105.16 cubic inches. The results give the average of three experiments.

MATERIALS EMPLOYED.	BUSHELS.	CUBIC FEET.
Cement, ...	3.40	4.36
Sand, ...	3.40	4.36
Bricks, ...	17.78	22.83
Totals, ...	24.58	31.55

It would thus appear that the shrinkage amounts to 14.4 per cent.; a volume of 31.55 cubic feet of materials forming 27 cubic feet in the work. From these experiments we may compute that a rod of brickwork (which contains 16½ feet × 16½ feet × 1⅛ feet, and has therefore a volume of 306¼ cubic feet, or 11⅓ cubic yards), formed with materials identical in every respect with the above, would require 38.5 bushels of cement, and 38.5 bushels of sand, and 4352 bricks, amounting together to 357.5 cubic feet as against 306.25 cubic feet in the work.

344. The practical applications of Portland cement are so numerous, that it may be said to have given to Civil Engineers and Architects a new building material rather than a new cement solely; and though it has not originated the use of concrete—which, when formed of suitable hydraulic lime, has done such good service in foundations and backing of retaining walls—yet it has widely extended its application by the confidence now felt in its superior strength.

The making of this cement has indeed almost reached the importance of a national manufacture; and as about one-third of its prime cost is due to the coal employed in the process, it must remain in the hands of those who possess a cheap supply of fuel. The following historical account of its applications has been taken from Mr. White's paper, vol. xi., "Proceedings, Inst. Civ. Engineers." Alluding to the method of building under water, by artificial blocks of concrete let down into place, he says:—

"Some account of the employment of Portland cement for concrete blocks in the Government works of this country and of France will afford the best criterion of its adaptation for such purposes.

"At the Government harbour of refuge works at Dover, under the superintendence of Messrs. Walker and Burges, these blocks are extensively used for that part of the hearting of the breakwater which, being under low-water mark, cannot be filled in with concrete *en masse*. The cubic contents of these blocks vary from 45 feet to 120 feet, and the weight from 3 tons to 7 tons, the dimensions of width and depth being generally equal, and the length about twice the width. The composition and the mode of construction are very simple.

345. "One cubic foot of cement, measured dry, is mixed with 10 to 11 cubic feet of shingle from the beach (consisting of large and small clean shingle and coarse sand in nearly equal proportions), in a box holding as much material as will form one block, the cement and shingle being spread in alternate layers until the box is full. One side of the box being then withdrawn, the contents are allowed to fall gradually on to the mixing platform, where water is added, and the concrete is made, which is then shovelled into moulds of the required size, in which it is allowed to remain about 24 hours, until it is sufficiently set to permit the sides of the mould to be removed. The blocks so made then stand for eight or ten days, and are then conveyed either into their positions in the work, or are more usually piled up for stock. The holes for the lewises penetrate two-thirds of the entire depth of the block, and are formed during the filling process; by inserting bevelled wooden plugs, which are withdrawn when the block is hard. The labour attendant on cutting the lewis holes, as in stone blocks, is thereby saved, and the risk of fracture is also diminished by the precaution of forming the holes nearly, if not quite, through the block. It will, however, give some idea of the cohesive power of the concrete to state that on the tenth day after being made, blocks of 3 and 4 tons weight have been lifted by lewises sunk only 8 inches into the mass.

"The great hardness and cohesive power of these blocks, and the success which has attended their manufacture (scarcely one per cent. of the whole number made having been broken), show this to be a suitable proportion for the blending of the materials."

The proportions, and the shrinkage, have been already given in page 303.

346. "At the harbour of refuge works of Alderney a similar application of concrete blocks is made as at Dover, with this difference, that in their manufacture rubble stone, which is very abundant in the island, is mixed with the shingle.

"The proportions adopted are one part of cement, two parts of sharp sand, and four parts of coarse and fine shingle;

these materials converted into concrete, and thrown into the moulds in quantities of 8 or 10 bushels at one time, suffice to form a layer over the whole surface of the mould. Flat pieces of rubble stone, varying in size from 3 inches to 20 inches in length, are then embedded promiscuously in the concrete; and this operation is repeated till the mould is filled. The quantity of stone thus incorporated in the block is found to constitute about 40 per cent. of the whole bulk, and the proportion of cement to the mass is 1 in 10, as at Dover. Lewis holes are formed as the work proceeds, and these blocks, like those made at Dover, are ready for moving in about ten days. They contain from 60 to 100 cubic feet, and their weight is from 4 to 6 tons, reckoning 16 cubic feet to one ton. The degree of smoothness and external finish given to the blocks depends chiefly upon the quantity of water used in mixing the concrete. If gauged soft, the cement will find its way to the sides and surface, and will give an even face to the concrete, which in time becomes intensely hard, and, as serving to protect the interior from the action of the sea when the blocks are used in the face of walls, it is, perhaps, preferable to leaving them with a rough and honeycomb face, the result of using but little water in mixing concrete. It is not, however, found needful to observe this precaution at Dover and Alderney, since at both places the blocks are used exclusively for the hearting, or for the foundations of the walls, and are consequently not in permanent contact with the sea and the shingle. [Above low-water mark the external face of the pier is formed of coursed aisler]. The blocks are set by the diving-bell at Dover, and by men in diving dresses at Alderney, and are bedded dry, their close contact with each other and their position in the wall rendering the use of cement unnecessary, except to point the joints externally as the work proceeds."

347. "The blocks made at the important works of the Digue, at Cherbourg, where Portland cement is largely used, are of enormous size, and are made in a different manner to that adopted at Dover or at Alderney.

"The dimensions of those made at Cherbourg are 12 feet long, 9 feet wide, and 6 feet 6 inches deep, giving a cube of 712 feet, or 52 tons. These masses are constructed like rubble masonry, without moulds, and are built up at low water in a position convenient to the work. When made they are slung to pontoons, and floated at high water to the point where it is desired to sink them, and then deposited '*a blocs perdus*,' at the extremities of the Digue, near to the upright wall, in order to break the force of the waves at its foot, and to prevent the disruption of the masonry. The object of making blocks of such immense size was to secure their remaining permanently where they are deposited, the calculation being that the waves would not have power to move masses exceeding 20 tons in weight. It has not, however, been found that even these immense monoliths can always resist the motive power of the waves; for, on more than one occasion, thay have been thrown up whole and unbroken from the bottom to the top of the breakwater wall, a height of at least 30 feet. The mortar used in the construction of these blocks consists of one part of Portland cement to three of sand, which materials unitedly occupy from 30 to 40 per cent. by measure of the whole mass.

348. "Another method adopted in this country of forming concrete stone blocks is to mix the cement with the stone in a state of grout—that is, mixed with a quantity of water sufficient to render it quite fluid. The stone is placed promiscuously in the moulds, and when one-third full, the grout, composed of one part cement and three parts sand, is poured in till the interstices are thoroughly filled up. It is true that blocks so made require longer time for induration than those made with cement stiffly gauged; but their ultimate solidity, and the thorough homogeneousness of the mass is evidenced by a specimen sawn from the heart of a block made of grouted cement, and Kentish ragstone, which has in twelve months acquired such hardness that the cement is scarcely distinguishable from the stone to which it adheres. In situations where gravel is scarce, and where time can be given for the induration of the blocks, this mode may be adopted with advantage.

349. The adhesive quality of this cement as mortar is seen by the following experiments by Mr. White to be of an intimate nature.

"Portland cement uniting two six-inch cubes of different kinds of stone, with a joint of from one-eighth to one-quarter of an inch in thickness; the surfaces cemented together being 36 square inches. The result given is the average of two or more trials:—

	Lbs. per sq. inch.
Portland stone, ...	146
Granite, ...	97
Bramley Fall, ... (Dolomite limestone).	76

"The mixture of four parts of sand with one part of cement forms a mortar which may be relied on in ordinary work, and is the proportion which has been used at the building of the Houses of Parliament. Less sand may, of course, be used; but where time is not an object, there seems hardly any limit to the quantity of sand which the cement will carry, provided only the grit be sharp and clean.

"In testing the resistance to compression of brick piers built with cement, the average resistance was found to be about 38 tons per square foot.

350. "It only remains to mention the application of Portland cement as a stucco. Its agreeable colour, which is susceptible of further improvement by the addition of any slight stone dust, or of white sand, joined to the peculiar property it possesses of resisting frost and the formation of vegetation, recommend it as an external covering. This power of resisting frost and the humidity of the atmosphere arises from the close contact of the particles, which produces a non-absorbent surface, and thus prevents the disintegration common to cement of less density, and renders it a peculiarly suitable material for lining reservoirs, cisterns, baths, &c., and has also caused its extensive adoption for the casting of ornamental figures, statues, fountains, &c., thus combining the appearance and durability of stone with the economy resulting from the use of the mould, rather than of the chisel, which, if not the most legitimate application of cement, is one which, by its increasing demand for objects of art, at a minimum cost, has been largely sanctioned and encouraged."

351. Under the head of resistance to direct compression, it has in very many recent works superseded, with great economy, the well known but expensive timber platform on bearing piles, and even cofferdams have been laid aside by the use of cylinders and rectangular caissons of cast and wrought iron, which, being open at the bottom and at the top, have been forced down through the ground to a firm base by loading the upper edge; the interior being excavated, either by divers or by scooping, and, on reaching a water-tight stratum, are pumped dry, and then filled with cement concrete for a depth of several feet, according to circumstances, upon which the superstructure is raised as upon a uniform bed of solid rock. At the Charing-cross Bridge, as much as 9 tons per square foot is sustained by the concrete within the cylinders.

In cases where it may not be possible to place this material on a dry bed, and where, consequently, the cement is liable to be washed out before setting commences, the concrete has been placed in sacks, which were laid, as soon as filled, under water, with some attempt at bonding, and in as many courses over each other as may be necessary to afford a surface upon which to place the loose concrete. It is found that the sacks, though somewhat cylindrical in shape, when lowered eventually fit closely, adapting themselves properly before setting to the form of the part in which they are deposited. This method has been adopted by Mr. Parkes in founding iron lighthouses upon coral reefs in the Red Sea ("Minutes of Proceedings, Institution of Civil Engineers," vol. XIII.). The holes in the uneven surface of the reef were filled up with concrete, charged into coal bags, and on the level surface thus obtained sheets of tarpaulin, covered with concrete, were lowered quietly. In this manner the soft material was protected from the action of the water until a mass of several tons was collected; a permanent caisson of wrought iron plates having been first placed around the site of the base of the lighthouse. The process of founding on concrete in sacks laid under water, has also been employed by Mr. B. B. Stoney with perfect success, in founding a quay wall 20 feet under low-water mark, on a loose gravel pervious to water.

352. As superseding the well-known and expensive timber platform on bearing piles for an artificial foundation, the use of concrete formed of Portland cement and gravel is now almost universal. Thus, at the Victoria Bridge, carrying one of the city lines over the Thames, we find the foundation for the footings of the piers thus described—after having mentioned the preliminary works of dredging the bed of the river, and enclosing the site of the piers with cofferdams extending 4 feet below the level of the intended foundation, the water being pumped out, and the clay excavated to the depth of 40 feet below high-water mark:—

"The space occupied by the foundations was next surrounded by permanent sheet piles driven to a depth of 8 feet below the foundation level. Within this sheeting a bed of cement concrete, 4 feet in thickness, was formed, and on that the masonry of the piers was commenced, the concrete being afterwards carried up to the top of the sheet piles round the entire circumference of the piers" ("Minutes of Proceedings, Institution of Civil Engineers," vol. XXVII., p. 58). The proportion was one measure of cement to six of clean sharp Thames gravel, dredged on the site of the foundations, which was screened and freed from excess of sand.

Subsequently this bridge was widened about 100 feet, and the spans adopted being, of course, identical with the existing bridge, the new portions of the piers were constructed by cast-iron cylinders 21 feet in diameter, sunk through the clay to a depth of 45 feet below high-water mark, entering 13 feet into the London clay; and the excavated interior filled with cement concrete for a depth of 12 feet the remainder, up to low-water mark, was built in with brick in cement. If the bridge, as completed, were loaded all over with locomotives, the greatest weight on the brickwork in the cylinders at low-water mark would be 6 tons per superficial foot, and upon the clay at the bottom 5 tons per foot, not deducting for the friction of the cylinders.

353. But it is not only in the work of securing a good foundation that this material is now employed. Retaining walls have in many places been constructed altogetherof concete without any facing of stone; the materials being mixed on the spot, are thrown by the shovel into place, and the proper form obtained by panelling of planks, which is easily removed as soon as that portion has set, and the same planks are ready for use again; one part of cement to ten of good gravel is found sufficient, large rubble stones being thrown in with it; and, as none but labourers are employed, the cost is found to be from ten to twelve shillings per cubic yard, which is about two-thirds of that of rubble work in lime mortar. The new and old portions of the wall unite perfectly, even when the work has not been carried on for a considerable time.

As a backing to a coursed aisler facing in dock and quay walls, mixed, in this case also, with large rough stone, a great economy in time and money has been effected, the lower part where exposed to any current of water being formed richer in cement than that above.

Mr. Grant, in his "Further Experiments on the strength of Portland Cement," has described the following practical applications—"In the construction of the southern Thames Embankment from the Westminster-bridge to Lambeth, it was originally intended to form the river wall of brickwork, with a granite facing; but after a fourth part of the work had been executed in this manner—a concrete was substituted for the brickwork, the front work being identical all through, and 14,355 cubic yards of Portland cement concrete, made in the proportions of 6 to 1, at 11*s*. per cubic yard, were substituted for an equal quantity of brickwork, at 30*s*. per cubic yard—a saving of about £13,600 on this part of the work: as to the prices above given, it is very probable that in this work the cost of carriage for the cement and the gravel could at no locality be less.

Sewers of considerable size are now formed of concrete of this cement. Everything being taken into consideration, the most economical combinatiom was found to be an interior lining of 4½ inches of brickwork in cement, and all the rest in concrete; in other instances sewers were formed entirely of concrete, the proportions being of one of cement to six of sand. The cost of this concrete was less than half that of brickwork; but if rendered inside with cement mortar (equal proportions of sand and cement), one inch thick, it was about the same as if lined with 4½ inches of brickwork. When sewers are formed of concrete without rendering or brick-lining, a long length of centering is required on account of the slow setting of the material, and about double the length of trench requires to be opened at one time, a disadvantage in busy thoroughfares.

The cost of a concrete sewer 4 feet by 2 feet 8 inches was 10*s.* per lineal foot, exclusive of excavation. Under the same contract a brick sewer of the same size, 9 inches thick, cost 16*s.* 6*d.* Another sewer was mentioned, 9 ft. by 9 ft., of concrete, with a lining of 4½ inches brick in cement. Sewers and culverts of almost any size might be made on this principle.

Mr. Grant also deduced the important conclusion that "concrete made of broken stone or broken pottery, was much stronger than that made of gravel, due, no doubt, partly to the greater proportion of cement absorbed in the latter case in cementing the finer particles of sand, and partly to the want of angularity in the gravel."

Mortar of Portland cement when mixed by hand, was compared with that mixed in a mortar mill for thirty minutes; the greatest strength of that mixed by hand was about double that mixed in a mortar mill. The former also maintained its strength; the mill-mixed declined from its maximum at a month to the end of the experiment. This result was probably due partly to the process of crystallization, or setting, having been interrupted by the continued agitation, and partly to the destruction by attrition of the angular form of the particles.

The numerous strikes and combinations of late years amongst masons, have made the introduction of Portland cement concrete more general and rapid, than it would otherwise probably have been. Thus in one case a Waterworks company having new filter beds to construct, in the design of which large quantities of brick walling were provided, found that the completion of the work was likely to be delayed by an impending strike, changed the design, and built the walls with a facing of brickwork half a brick thick, filling in the back with concrete, formed of the screenings of the sand for the filter beds, mixed with flat rubble stone; at every 10 or 12 courses, a course of headers was introduced which bonded the front work to the backing. These walls were constructed with great rapidity and economy, and have proved perfectly successful.

Portions of works below low-water, in which cofferdams have not been employed, have been successfully founded by pouring down, at still water of the spring tides, large masses of several tons, mixed with a very large proportion of cement, which provides for any loss of cement which may be washed away in falling through a few feet of water. Subsequent opportunities of examining portions of concrete thus deposited have proved it to be quite successful. The operation is thus described in a Paper by Mr. J. J. Mann, assistant to Mr. B. B. Stoney, Engineer to the Dublin Port and Docks Board, on the "Repair of the River Wall at Burgh-quay, Dublin." (Trans. Inst. Civ. Engineers of Ireland, vol. ix., p. 23):—"All the soft mud and sand in the space between the front sheet piles and the wall, to a depth of 2 feet below low water, was removed, and replaced by Portland cement concrete, well rammed. The concrete was formed of four parts clean ballast to one of cement, thoroughly mixed in as stiff a condition as possible, and put in place at dead low water of spring tide. The propriety of placing concrete in water, even 3 feet in depth, as was done in the present instance, without any protection to obviate the cement from washing out, has frequently been questioned. There is little doubt, however, that if Portland cement concrete is mixed extremely stiff, it may be thrown into moderately still water of the depth specified, without suffering any serious injury."

On subsequently rebuilding part of the opposite quay wall this operation was extended to greater depths; masses of 8 cubic yards, at one time, were launched at low water down an inclined plane of planking into a space dredged out between the lower parts of the old wall and the new lines of sheeting piles in front, which were cut off at low water; and upon the bed of concrete so formed a heavy quay wall was erected. The scarcely altered colour of the water in which it was immersed proved that the cement was not washed out in any injurious degree. The slow and expensive process of lowering the concrete in boxes with a moveable bottom, containing only a few cubic feet, may therefore, with judicious management, be dispensed with.

The use of Portland cement concrete in the erection of the walls and partitions of dwelling-houses, which has for several years been proposed, seems at length to be confirmed as one of its most successful applications. The desire to provide healthier, cleaner, and more decent dwellings for the working classes, which has been so long disappointed by the seeming impossibility of combining a fair interest on the capital invested, with the improved accommodation required, is now accomplished. The economy in the use of concrete in lieu of ordinary building material, will depend on the facility with which gravel, sand, and small broken rubble, can be obtained; and in some degree also on the expense of carriage of the cement. Under favourable circumstances the cost may be set down as one-half that of other modes of building, the diminished cost arising not only from the facility of forming doorways, reveals of windows, and flues, but also from the lessened expense of plasterer's work, one coat alone being used. The thickness of the main walls is generally only 9 inches, internal partitions 4½ inches.

Bridges of large size have been formed altogether of concrete, somewhat in the manner described above for retaining walls, the laggings of the centering being laid close and smoothly wrought, so as to give a good soffite.

The foundation of roads has been attempted with it, and many other applications are proposed, for which reference may be made to "Reid, on the Manufacture of Portland Cement," and the same author's work, "Practical Treatise on Concrete."

354. The strong adhesion of this cement to iron is made use of by iron shipbuilders, who now line the interior of the bottoms of vessels with a coating of cement mortar, about 4 to 6 inches thick, formed of about two of sand to one of cement, and even the channels on each side of the deck leading to the scupper holes are sometimes lined in the same manner. The interior of cast-iron cylinders laid at a gradient, as sewers, has also been lined with cement mortar between the internal flanges, thus saving the iron from the injurious action of the sewage and affording a smooth surface for its flow.

It is important to observe that Mr. Grant is, in his second Paper already quoted, enabled to say—" The quality had not only been maintained, but had continued to improve." The specification has now been increased to 787 lbs. tensile resistance on the area of 2¼ square inches, at seven days, which is equal to 350 lbs. per square inch, a stipulation much below the average results actually obtained during the last five years—namely, an average tensile strength of 806·63 lbs. from 589,000 bushels, equal to 358.5 lbs. per square inch, after seven days. The strength at the end of thirty days of 37,200 bushels of the above cement, as ascertained by 1180 tests, averaged 1024 lbs., equal to 455 lbs. per square inch, a result 30 per cent. higher than that at seven days. " Whenever the nature of the work would admit of it, tests at the end of a month would be found more satisfactory than if made earlier, as heavy cements, though the strongest eventually, were the slowest to set."

If we collect from p. 296 what has been given as the earlier specifications on the main drainage, and add the latest, we have the following statement, the test being at seven days, and the weight 110 lbs. per bushel, or 85.7 lbs. per cubic foot:—

Date.	On 2¼ square inches.	On 1 square inch.
	Lbs.	Lbs.
1st, 1859,	400	177·8
2nd, 1862,	500	222·2
3rd, 1864,	562·5	250
4th, 1871,	787	350

Further experience has confirmed the earlier conclusions, that the strength of Portland cement increased with its specific weight, its more perfect pulverization, and its thorough admixture with the minimum quantity of water in forming mortar. Roman cement, though only two-thirds the price of Portland, was but one-third of its strength, and therefore double the cost when that is measured by strength; its sole advantage lying in its quickly setting, which is valuable for many purposes.

Mr. F. Ransome has applied this cement in the manufacture of his " patent concrete stone." The materials employed being " ordinary sand, Portland cement, ground carbonate of lime, and soluble silica, well amalgamated, and mixed with a sufficient quantity of his ordinary silicate of soda;" thus obtaining a plastic mass, which is capable of being moulded, and after a short time assumes a great degree of hardness, strength, and density. Its power also of resisting injury from heat, cold, and moisture, with its freedom from unsightly efflorescence, will eventually give it a high place amongst constructive materials.

355. The importance of STEEL, as a material for construction in works of civil and mechanical engineering, is now so generally acknowledged, that, placing it last in any enumeration of the strength and applications of the materials used in practice may seem to require an explanation; but the earlier portions of this work had been long in print before the great discovery of Henry Bessemer was brought into use. Until the complete success of this invention, the employment of cast-steel was confined to articles of very small size, such as cutlery, edge-tools, &c., the process of manufacture being, in truth, but little changed since Huntsman, in 1740, made his most valuable discovery.

356. A very brief account of the mode of manufacture in each case may best show the contrast between them—and, first, of the crucible cast-steel. In this method of making cast-steel, the crude pig-iron has first to go through all the stages of melting, refining, puddling, hammering, and rolling, in order to produce a bar of malleable iron as nearly pure as the most careful selection of ores and manipulation in charcoal fires can make it. Bar-iron, on which so much labour, fuel, and engine-power have been expended, thus becomes the raw material of this most expensive manufacture. In order to convert the wrought-iron bars above described into blister steel, they are packed with powdered charcoal in large fire-brick chests, and are exposed to a white heat for several days, a process technically called cementation, the time required for heating and cooling them extending over a period of fifteen to twenty days. When thus converted into blister-steel—so called from the blistered appearance the surface of the bars often presents—they are then broken into small pieces, and sorted according to the quality of the steel, which sometimes differs even in the same bar. For melting this material, powerful air furnaces are employed, containing two crucibles, into each of which are put about 40 lbs. of the broken blistered-steel. In about three hours the pots are removed from the furnaces, and the melted steel is poured

into iron moulds, and formed into ingots of cast-steel; in some cases the contents of the crucible are poured directly into a mould, and given the required shape at once; from 3½ to 4 tons of hard coke being consumed for each ton of metal thus melted. For large masses of metal, a great many crucibles must be prepared all at the same moment, and a continuous stream of the melted metal from the several crucibles must be kept up until the ingot is completed, since any cessation of the pouring would entirely spoil it: hence, in proportion to the size of the ingot, are the cost and risk of its production increased.

Thus, quite recently, at the works of Messrs. Vickers and Sons, of Sheffield, in forming a steel crank axle, cast to shape, and weighing when finished 17 tons 4 cwt., there were used 550 crucibles, the contents being poured in regular order. Each must, therefore, have contained 70 lbs. on the average.

357. The bars of blistered steel are also used for several purposes, without being melted in crucibles. From the nature of the process the bars are hardest outside, and therefore unfit for immediate use, except for a few purposes, such as files, shovels, &c.

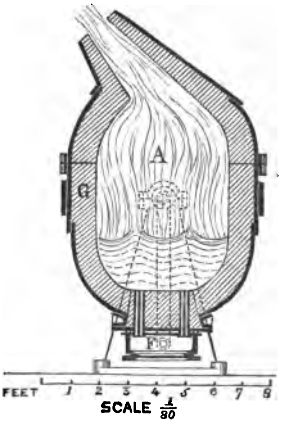
358. Shear-steel is blistered-steel cut up, piled, reheated, and welded under the tilt-hammer, the piling answering the same purpose as fusion, in producing homogeneity of structure, but less perfectly.

Double shear-steel is single shear-steel a second time cut up, piled, heated, and tilted.

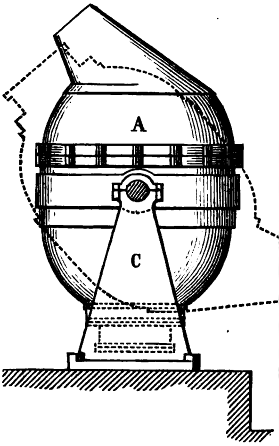
If cast-steel, therefore, were ever destined to supersede wrought iron for general purposes of construction, it was necessary to cease employing wrought iron, and that of the highest quality, as a raw material for this otherwise most expensive mode of manufacture; and this great object has been practically and commercially attained by the process of Mr. Bessemer, which has produced a complete revolution in this branch of metallurgy, not only reducing the cost of manufacture by more than 50 per cent., but enabling the quantity that may be produced to keep pace with the great consequent demand which has arisen.

359. The following description is from the words of the inventor, in a paper read before the Institution of Mechanical Engineers, at Sheffield, in 1861, first referring to the wood engravings showing the construction of the converting vessel:—

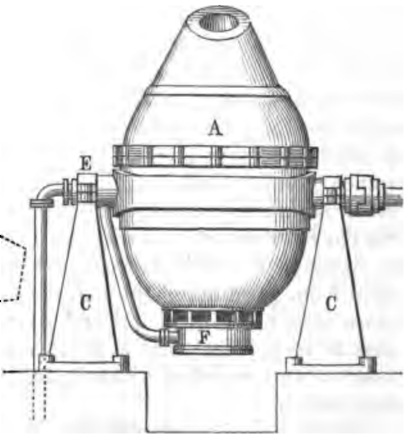
In each engraving A is the "converter," which is formed of boiler plate, and revolves on its trunnions in bearing formed on the top of the cast-iron standards C; one of these bearings, as at E in the front elevation, is hollow, and through it the compressed air from the blast engine is conveyed to the small chamber F, entering the interior of the converter through fire-brick twyers, as shown in the vertical section, which have 49 holes, four being cut through in the vertical section—namely, 7 in each of the seven distinct twyers, firmly fastened to the base of the converter. The lining of ganister is lettered G in the vertical section. The upper part of the vessel, at the letter A, is bolted down to that which carries the axes or trunnions, and the air chamber F is also bolted on below, so that the interior lining of ganister and the twyers can be renewed when necessary. The object of giving the vessel the power of revolving is for the purpose of turning it into such positions as to receive the charge of molten iron without entering the twyers, and for pouring it, after conversion, into the ladle, as in the dotted lines in the side elevation; it can even be completely turned down to let all slag or scoriæ drop out.



Vertical Settion.



Side Elevation.



Front Elevation.

"The crude pig-iron used in this process is hot blast hæmatite, smelted with coke, which is melted in a reverberatory furnace adjoining, and is then run into the converting vessel (shown in the wood engravings). This vessel is made of stout boiler plate, lined with "ganister," which is a powdered siliceous stone found under the coal, in the neighbourhood of Sheffield. It costs but 11s. per ton, and is both cheaper and much more durable than fire-brick. The blast from the engine is conveyed through one of the bearings, and enters the tuyeres at a pressure of 14 lbs. per square inch, which quite prevents the fluid metal from entering. Before commencing the first charge of metal, the interior of the vessel is heated by coke, with a blast to urge the fire. When sufficiently heated, it is turned over, and all the unburnt coke falls out. The vessel is then turned into the proper position and the melted pig-iron is run in from the furnace by a spout, the vessel being kept in such a position that the tuyeres are above the surface of the metal. When the proper charge is in, the blast is laid on, and the vessel quickly turned into its vertical position. The blast now rushes upwards from the 49 holes in the tuyeres, producing most violent agitation in the whole mass. The silicium is first attacked, uniting readily with the oxygen of the air, forming silicic acid ($Si\ O_3$), at the same time a portion of the iron becomes oxidated, hence a fluid silicate of oxide of iron is formed, some carbon being, at the same time, burnt off. The heat is thus gradually increased till all the silicium is oxidised, which takes place in about twelve minutes from the commencement of the process. The carbon of the pig-iron now begins to unite more freely with the oxygen of the air blown in, producing first a small flame, and in about three minutes intense combustion is going on. The metal rises in the vessel, sometimes occupying double its former space, and in this frothy state it presents an enormous surface to the air, the oxygen of which unites rapidly with the carbon of the crude iron, producing intense combustion. The whole mass is now a mixture of metal and flame; the combustion of the gases is so perfect, that a great white flame rushes from the mouth of the vessel, indicating by its colour the nature of the metal inside.

The blowing is left off when the length of time from the commencement and the appearance of the flame indicate the required quality of metal, and that the proper per centage of carbon is left. At least this is the mode of working the process in Sweden. But at the works in Sheffield, it is preferred to continue blowing till the flame suddenly drops, which is just as the metal approaches the state of malleable iron, all the carbon in the charge being burnt off by the oxygen of the air driven through. The blast is now turned off, the vessel is tipped forward, a small measured quantity of artificially prepared charcoal pig-iron, *speigel eisen*, from 120 to 140 lbs. to a charge of 20 cwts., containing a known proportion of carbon, and from 8 to 10 per cent. of manganese, is then added; when the vessel has been righted the blast is continued for a few seconds.

Thus steel of any desired degree of carburation is produced, the process having occupied altogether about 28 minutes.

The vessel is then turned into the position shown dotted in the diagram, and the steel is run into a ladle, from which are filled the ingot moulds. By tapping the metal in this ladle from below, no scoria or other *floating* impurities can run into the mould, and the fluid steel is dropped straight down the centre of the mould without touching the sides. From one to ten tons of crude iron may be converted into cast steel in thirty minutes, without using any fuel except that required in melting the pig-iron and in heating the vessel.

The loss on the weight of crude iron is from 14 to 18 per per cent., with English iron worked in small quantities; but with purer iron in Sweden the loss is only 8.75 per cent.

The largest sized apparatus at present erected has a vessel capable of converting 4 to 5 tons at a time, which it converts into steel in 28 minutes.

In consequenee of the size of these vessels, no metal is thrown out during conversion, and the loss of weight has fallen as low as 10 per cent., including loss in melting the pig-iron."

360. But one point needs to be added to the above account of this novel and extraordinary method of producing steel, which is, that only those ores of iron which are nearly free from phosphorus and sulphur can be employed in the production of the pig-iron to be used in the converter: unlike the puddling furnace which, in the manufacture of wrought-iron, to a very great extent deprives the charge of these injurious impurities, the converter of Mr. Bessemer fails to act upon them, and thus the necessity arises for the sole use o ores which, like the hæmatite, are found to be naturally fres from these elements; as in the older methods, therefore, thi process depends for complete success on the use of the best qualities of cast-iron as a raw material. The ores of the great

iron district of Cleveland which contain about 1·15 per cent of phosphorus cannot be used, and in like manner the ores of the districts of Staffordshire and South Wales are unsuited to this manufacture. The largest works in England are, therefore, found at Barrow-in-Furness, on the coast of Cumberland near the great deposits of hæmatite ores: it is true that they have the disadvantage of importing their coal by rail from Durham. The manufacture is also carried on in the great coalfields, the ores being carried to the fuel. It has been stated that the percentage of phosphorus which may be present in any ore for steel making should not exceed 0·05 per cent.

The fame and fortune which have so deservedly rewarded Mr. Bessemer have stimulated other metallurgists, and already several new processes are before the world: that of Heaton proposes to use the ordinary pig-irons which are employed for wrought-iron, purifying them by the use of nitrate of potass: the Siemens-Martin process, and others, on the success of which time will give the fairest verdict.

361. Before giving a distinct proposition stating the ultimate tensile and compressive resistance of Bessemer Steel, as has been done in the case of iron and timber in the preceding pages, it is advisable to state that the qualities of hardness, ductility, and cohesion have been proved by experiment to be directly connected with the percentage of carbon contained in the specimens under trial; and that by skill in the selection of certain proportions of the materials in the first place, and subsequent manipulation, any desired result may be obtained. It is generally stated that wrought-iron contains under 0·5 per cent of carbon, steel from about 0·33 to 1·5 per cent., and cast-iron from 1·5 to 4, (or even more), per cent. It appears, therefore, that the range is limited, though such very wide differences in strength, hardness, &c., result from changes in the carbon present.

362. Mr. Vickers, of Sheffield, has experimented upon different qualities of steel (Proceedings of the Mechanical Engineers for 1861, page 158 to 170), the specimens being turned down to one inch in diameter for a length of 14 inches, with the following results:—

Carbon per cent.		Ultimate Tensile Resistance per sq. inch.	Extension in inches.
		Tons.	
1st. Experiment,	.. 0·33	30·4	1·37
10th do.,	.. 1·25	69·0	0·62

The ultimate strength was thus more than doubled; but the extensibility reduced more than one-half. Eight intermediate experiments gave corresponding results, the increase of carbon giving increased strength and shewing at the same time a diminished elongation. Beyond 1·25 per cent. of carbon, the steel becomes gradually weaker until it reaches the form of cast-iron, which we know has the very low tensile resistance of 6 to 7 tons per square inch. If, therefore, a mild tenacious material, free from brittleness, capable of resisting concussion, and having the toughness and ductility of wrought-iron, be desired, the percentage of carbon must be small.

363. The following Table has been arranged from the summary of experiments, by Mr. Fairbairn on steel from the Barrow Company, whose works are devoted to the production of steel suited to girders, boilers, rails, and other purposes of construction.

The testing machine was not capable of giving a higher compression than 100·7 tons per square inch.

364. **Bessemer Steel.**—*The Ultimate Tensile Strength of mild Steel, adapted to purposes of construction, is about* 38 *tons per square inch of Section on an extended average of experiments.*

365. TABLE, *showing the Ultimate Resistance of Steel, cast from the Bessemer process, to forces of Tension and Compression; Specimens supplied by the Barrow Hæmatite Steel Company.*

TENSILE FORCES.				COMPRESSIVE FORCES.		
No.	Specific Gravity.	Breaking Weight per square inch.	Elongation length of Specimen = 1.	Greatest Weight per sq. inch.	Compression per unit of length.	
		Tons.		Tons.		
1.	7·7006	41·70	0·0406	100·7	0·200	No cracks.
2.	7·7710	36·03	0·0866	,,	0·450	,, ,,
3.	7·7899	30·63	0·0656	,,	0·450	,, ,,
4.	7·7037	29·59	0·1858	,,	0·480	,, ,,
5.	7·7978	32·69	0·0312	,,	0·525	,, ,,
6.	7·7952	30·69	0·0812	,,	0·474	Slight cracks.
7.	7·7956	33 81	0·0906	,,	0·392	No cracks.
8.	7·8654	33·04	0·0765	,,	0·400	,, ,,
9.	7·8159	33·53	0·1000	,,	0·400	,, ,,
Average,		33·52	0·0842		0·419	

The length of the specimens employed for determining tensile resistances, was such that the elongations could be measured on 8 inches, the diameter at the neck being, practically, ¾ inch. The height of the specimens for compressive resistance was one inch, and the diameter 0·72 inch.

The same author has also published the results of a very extended series of experiments on Bessemer Steel, both as to its tensile and compressive resistances. The following Table has been arranged and condensed from that which gave the ultimate tensile resistances; the names of the several manufacturing firms being omitted and replaced by capital letters for purposes of reference. The figures represent tons per square inch.

366. TABLE, *showing the Ultimate Resistance of Bessemer Steel to a Tensile force.*

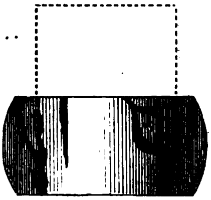
	Lowest Tons.	Highest Tons.	Average Tons.
A. . .	26·57	51·86	40·36
B. . .	36·37	53·75	45·15
C. . .	39·58	59·87	48·25
D. . .	38·26	55·21	46·87
E. . .	35·09	46·02	40·16
F. . .	33·57	48·18	42·65
G. . .	32·70	47·42	41·60
Average,	34 59	51·76	
	General Average.		43·46

The mean lowest being thus about 67 per cent. of the mean highest. The average of both Tables § 360 and 361 being, as stated in the proposition, 38·5 tons per square inch.

It is abundantly evident from what has now been given, that the ultimate tensile strength of Bessemer Steel is about double that of ordinary wrought-iron of good quality.

367. The following more detailed statements of experiments on the compressive resistance of steel, as given by Mr. Fairbairn at Dundee, in 1867, have been selected from the complete accounts of each separate trial together with a copy of the engraving that accompanied each, showing the appearance of every specimen under the ultimate load, the dotted lines showing the original dimension.

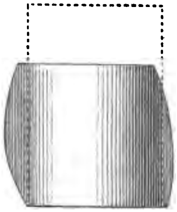
Exp. VIII. Bar of steel from Messrs. John Brown, and Co., marked, "B. S.,"—described as "Foreign Bar, tilted."



	Before Experiment.	After Experiment.
Height of Specimen,	0·989 inches, .	0·497 inch.
Diameter ,,	0·72 ,, .	0·886 ,,
Area, . ,,	0·40715 sq. in. .	0·61653 sq. in.

Specimen very much cracked. Ultimate compression, 0·493 inches.

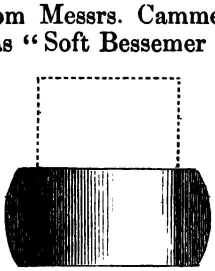
Exp. XI. from Messrs. Cammell and Co. Mark on bar, "2."—described as "Tool Steel."



	Before Experiment.	After Experiment.
Height of Specimen,	1·005 inches, .	0·749 inch.
Diameter ,,	0·72 ,, .	0·772 ,,
Area of ,,	0·40715 sq. in. .	0·46808 sq. in.

Ultimate compression 0·263 inches. No cracks.

Exp. XV. Bar from Messrs. Cammel and Co. Mark on bar, "6."—described as "Soft Bessemer Steel."



	Before Experiment.	After Experiment.
Height of Specimen,	0·997 inches, .	0·514 inch.
Diameter of ,,	0·72 ,, .	0·891 ,,
Area of ,,	0·40715 sq. in. .	0·63334 sq. in.

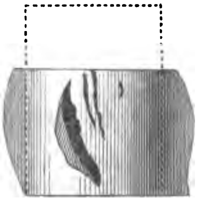
Ultimate compression 0·493 inches. No cracks.

368. In this experiment, as being on a specimen of soft Bessemer Steel, the following further details are quoted:—

	Weight laid on in Tons per sq. in.	Compression in Inches.
No. 1.	41·049	0·080
2.	49·303	0·135
3.	57·198	0·203
4.	64·637	0·269
5.	72·391	0·328
6.	80·233	0·387
7.	87·952	0·426
8.	96·636	0·465
9.	100·700	0·493

It is surprising to observe that although the length of the cylinder was diminished by one-half no crack ensued, proving that this quality of steel would be safe from sudden fracture when exposed in any structure to concussions or vibrations.

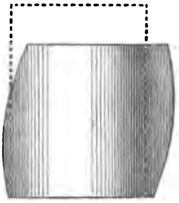
Expt. XXXV. Bar from Messrs. Saunderson and Co. Mark on bar, "S. 5."—described as "Drawn Bar."



	Before Experiment.	After Experiment.
Height of Specimen,	1·01 inches, .	0·678 inches.
Diameter of ,,	0·72 ,, .	0·790 ,,
Area of ,,	0·40715 sq. in.	0·49016 sq. in.

Commenced to crack at 80·233 tons per sq. inch. Ultimate compression 0·333 inch.

Expt. XL. Bar from Messrs. Turton and Sons, Mark on bar, "D."—described as "Steel used in the manufacture of machinery."



	Before Experiment.		After Experiment.
Height of Specimen,	1·00 inches	.	0·77 inches.
Diameter of ,,	0·72 ,,	.	0·76 ,,
Area of ,,	0·40715 sq. in.		0·45364 sq. in.

No cracks. Ultimate compression 0·233 inches.

369. The same difficulty in determining the true ultimate resistance to compression arises with these tough and ductile qualities of steel, as with wrought-iron. Mr. Fairbairn calls it 100·7 tons per sq. inch in the ratio he assigns to the compressive compared with the tensile resistance; but this weight seems to have been the utmost power of the testing machine, rather than that of the specimens. However a material which as in § 368, is found to resist upwards of 40 tons per sq. inch with a compression of about 8 per cent., must be considered to hold a very high place among the materials at the command of Engineers.

370. Mr. George Berkeley has made a very valuable series of experiments on the compression of Bessemer Steel. The specimens were all cylinders, 0·755 inches in diameter, and, therefore, 0·488 square inches in area, the length 1·033 inches, some few being one more or less in the place of thousandths. Minutes of Proceedings Institution Civil Engineers, Vol. XXV., page 385–391.

There was, it will be observed, a remarkable uniformity in the resistance of the specimens up to 18,000 lbs. on the actual surface 0·755 inches in diameter, which is equal to 17·94 tons per square inch, after this weight the length gradually diminished.

371. Comparing this result with that of the compression of wrought-iron given in page 227, we may safely infer that the steel may be assumed to have nearly double the strength of wrought-iron. It was, however, in favour of the steel that the specimens had only 1$\frac{1}{3}$ diameters in height, whereas Mr. Hodgkinson's specimens of wrought-iron had a height of 4 diameters.

372. TABLE, *showing the Compression*[*] *Resistance of Bessemer Steel, the Specimen being cut from Railway Bars.*

No. of Expt.	Length.	Ultimate Length.	Amount of Compression.	Ultimate diameter.	Weight per square inch.
	Inches.	Inches.	Inches.	Inches.	Tons.
1	1·032	0·8380	0·1940		59·8
2	1·033	0·563	0·470	1·09	99·65
3	1·033	0·564	0·469	1·062	99·65
4	1·031	0·538	0·493	1·086	99·65
5	1·032	0·548	0·484	1·077	99·65
6	1·033	0·535	0·498	1·09	99·65
7	1·033	0·543	0·490	1·033	99·65
8	1·033	0·546	0·487	1·08	99·65
9	1·033	0·573	0·460	1·05	99·65
10	1·033	0·562	0·471	1·06	99·65

373. One of the most important items in the cost of railway maintenance is the renewal of rails, which has amounted on lines, which, like the North-Western and Great Northern, have a large and concentrated traffic worked at a high speed, to about £150 per mile per annum, on an average of nearly 17 years. On other main lines this may be put at about £70 to £80.

No ordinary wrought-iron material, or method of piling or making the finished rail, will however resist the crushing action of modern locomotives, which are not seldom constructed to work with 11 tons on the driving wheels. Amongst the most important of the methods hitherto used for this purpose, is that of forming the wearing surface of the rails entirely of steel, by introducing a bar of steel into the pile, and rolling it out, so as to unite it with the wrought-iron body of the rail. Another method is to submit the surface of the ordinary iron rail to a process of conversion in a furnace especially adapted, thereby case-hardening the outer coat or skin of the wearing portion of the rail. Both of these processes to a certain extent fulfil their object ; still they are open to the serious objection that only the crust or skin of the rail is rendered hard, and they do not prevent the body of the rail from yielding to the severe pressure of the wheels; lamination and splitting are only to a small extent diminished, and though the life of the rail is prolonged, the prolongation is uncertain. The same objection applies to the puddled-steel rail, and it is also liable to vary considerably in its hardness, and to be at times too brittle for perfect safety. This liability to vary in quality is inseparable from the mode of manufacture, and though many goodrails of this kind have been produced, the want of certainty in the process diminishes its value.

374. The introduction, however, of Bessemer's system has opened out a mode of producing a pure, homogeneous, hard, and tough material, most admirably suited for the manufacture of rails. Their cost, it is true, is higher; at the present time about £10 10*s.* to £12 per ton for steel, compared with £6 10*s.* to £7 for wrought-iron. No doubt on every railway of large traffic there are certain places where they could be laid with economy; for instance, where the traffic is so constantly severe that the ordinary points and crossings have to be renewed, it may be even four times a year. Once laid of steel rails, they would give no trouble for many years.

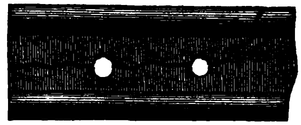
Two steel rails of the double-headed pattern were laid in May, 1862, at the Chalk Farm Bridge, near the entrance of the London and North-Western Railway into the Camden Town Depôt, side by side with two ordinary iron rails. The steel rails were taken up in August, 1865 (three years and three months); it was found at the end of that period that the upper surface, which alone had been exposed, was evenly worn to the extent of little more than a quarter of an inch, and still appeared to be capable of enduring much more work; and having carried the traffic of 9,550,069 engines, trucks, &c., and 95,580,000 tons. Meanwhile the companion ordinary wrought-iron rails, on that same line of way had been renewed eight times, each rail being once turned; thus giving as a result sixteen faces of wrought-iron worn out, to one of steel, and this latter not destroyed or incapable of further use on the line.

375. The ingots, which have been mentioned in § 354, are cast of such weight and form as are necessary for the production of each rail. Thus for a six yard rail of 84 lbs. per yard, the ingot requires to be 9 inches square, and 26 inches long. This ingot is hammered down to 6 inches square, and 5 feet long, and then rolled in the ordinary way. It will be evident that the only limit to the length of the rail made in this simple manner, is either in the weight of the ingot, which can be produced, or in the length of the rolling mill or heating furnace. It is as easy to produce long

lengths as short ones; and in this respect the above method has some advantage over piling.

There is no tendency to lamination in this perfectly homogeneous material; its toughness and durability have been fully proved by specimens, which have been twisted and bent while cold, without exhibiting any fractures; and the experience obtained from its use for several years past justifies all that has been expected from it.

376. In the opinion of several experienced engineers, the very rough test applied to wrought iron rails is quite inapplicable when they are made of Bessemer steel; every individual rail should be tested when made of this material; and, if this be so, it is evident that the tests applied must be such as will leave the rail uninjured, and yet prove it fully up to the greatest load it can be called upon to bear in its after work on the line. A testing machine answering these requirements has been invented by Mr. James Price, and described by him (Vol. xxxii., Minutes of Proceedings Inst. Civ. Engineers). Although Bessemer steel exhibits in the highest degree all the qualities of a good rail—namely, strength as a girder to sustain a moving load between the supporting chairs; secondly, toughness, to resist sudden strain or impact; thirdly, solidity, to resist separation under pressure; fourthly, hardness, to resist wear of the surface—yet a certain dangerous brittleness is found, at times, in a few rails out of large numbers of excellent quality, manufactured in the same manner, and by the same makers; these brittle rails, as they break up into several fragments, are more dangerous than the same proportion of bad wrought iron, and are not to be distinguished by their external appearance.



377. The wood-cut exhibits the manner of fracture of some steel rails used on the Cologne-Minden Railway, it was found that a piece, about 15 inches long, frequently broke off at the end of the rails. The fracture passed in each case through the two holes for the fish bolts. The oval holes were formerly punched, but they are now made by drilling, and subsequent enlarging by the slotting machine; it has not been ascertained, however, whether this will prevent the fracture just described. (Engineering, Vol. xii., p. 295).

In the formation of rails, wheel tyres, and axles for railway carriages, this steel may be considered as perfectly successful; and it has been applied, as rolled plates, to boilers also; but in this case, as in that of ship building, and girders, it has not as yet been very generally adopted.

378. The following Table, collected from the works of Tredgold, Rennie, &c., exhibits at one view the properties of four metals which are indispensable in practical construction, though not so much used, as those already discussed, in directly resisting compression and tension. Indeed, the ultimate resistance to compression of the more ductile of these cannot well be ascertained; it is only possible to record the successive diminutions of length and the corresponding resistances which at length become enormous.

The fraction given in the sixth column of the Table indicates the diminution of length, which had taken place in the specimens when the weight recorded at the right hand side was acting upon them.

379. TABLE *showing the Properties of the several Metals, Lead, Copper, Zinc, Tin, and Alloys.*

	Resistance to Tension. Tons per square inch.	Specific Gravity.	Weight per cubic foot.	Weight of a bar 1 foot long and 1 inch square.	Resistance to compression Tons per square inch.
			Lbs.	Lbs.	
Lead, ...	1·50	11·353	709.5	4·94	$\frac{1}{2}$ 0·8625
Copper—Wrought, sheets. ,, Cast, ...	13·50 8·5	8·7	549	3·81	$\frac{1}{16}$ 6·12 $\frac{1}{8}$ 11 50 15·07
Zinc, ...	3·50	6·96	431	3·00	
Alloys of Copper and Zinc, Brass, 2 Cu + 1 Zn,	12·5	8·3	519	3·6	$\frac{1}{10}$ 5·74 18·40½
Tin, ...	2·5	7·3	455·7	3·165	$\frac{1}{16}$ 0·986 $\frac{1}{3}$ 1·725
Alloys of Copper and Tin, Gun Metal, ...	16·10	8·56	535	3·715	
Hard Brasses for bearings, &c., in Machinery, . .	13·6	8·73	545·5	3·79	

380. The tensile resistance of lead is very important in the case of water pipes under great pressure. Copper bolts in the fire boxes of locomotive engines are under considerable strain from the steam pressure, and the importance of having them made of wrought, and not of cast copper, is evident. Gun metal, in that application which gives it this designation, namely in Artillery, requires all the tensile power and other qualities with which it has so long been credited.

APPENDIX.

No. I.

ON THE BEST FORM OF WROUGHT-IRON LINKS OR BARS, WHEN RESISTING TENSILE STRAIN.

SINCE the examples on wrought-iron bar links, given in pages 39 to 45, were printed, two valuable papers on the subject have been published, one by Sir Charles Fox, "Proceedings Royal Society," 1865; the other by Mr. Berkeley, vol. xxx. "Minutes of Proceedings Inst. Civ. Eng." The former had been entrusted by Mr. Vignoles with the manufacture of the links of the suspension bridge intended to span the Dneiper at Kieff, in Russia. The total weight of the bars was about 1600 tons, and, as they had to be shipped from Odessa, and afterwards conveyed by land carriage, over bad roads, for about 300 miles, to their destination, it became a question of great interest to determine whether the form of link designed was of the very best proportions, and equally strong in every part, so that no superfluous weight should have to be carried either on the journey to Kieff, or when permanently erected in the bridge. It is strange that we owe these valuable experiments, seemingly, more to the former than to the latter, and vastly more important, consideration.

The links were 12 feet long, from centre to centre of eyes—a length designed with a view to facility of transport; for, compared with other suspension bridges, in which a length of 20 or 24 feet has been adopted, the number of joints, with the necessary pins, caps and screws, &c., is disadvantageously increased, in equal spans, as the links are made shorter. The iron used was of high quality, portions one inch square, in section, having, from a number of experiments, given an ultimate tensile strength of 27 tons. The wood engraving (Fig. 1), at a scale of ¾-inch to one foot, shows the form and dimensions of the head of the link as first designed; the pin 4½ inches diameter; breadth on each side, 6 inches; the transverse section of the bar or shank being 10¼ square inches, the thickness in all being one inch. If, therefore, it had yielded in that part, it should have borne a little more than 270 tons; a proving machine, capable of exerting that force, was therefore provided. The transverse section of the pin, 4½ inches in diameter, being 15.9 square inches, or 50 per cent. greater than that of the bar, it is evident that no shearing of the pin can take place. This link gave way, however, under a strain of 180 tons, by tearing across at the eye, as shown in Fig. 4: proving at once the great disproportion between the head and the shank. It was determined, therefore, to increase the dimensions at the part which was supposed to be the weakest, and a few links were prepared, shown in fig. 2, having the same diameter in the pin, but with a breadth of 7 inches on each side of the

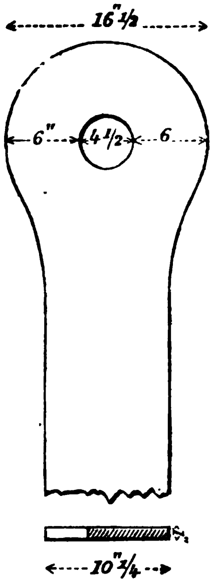


Fig. 1.

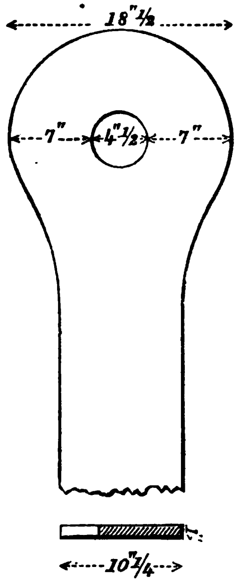


Fig. 2.

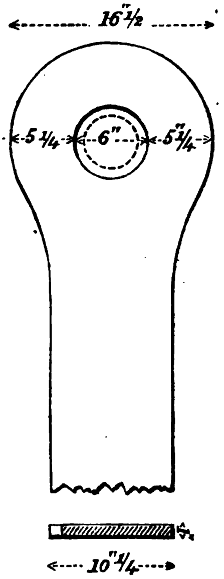


Fig. 3.

pin-hole, instead of 6 inches, as in fig. 1. It was surprising to find that this bar did not sustain more than the former, and broke at the same place, and in a similar manner, across the head. The cause of fracture was, therefore, not yet ascertained, but, on attempting to adjust the piece broken off to the position it originally occupied (fig. 4), it was observed that, while the fractured surfaces came in contact at the outside of the head, they were a considerable distance apart at the edge of the pin-hole; and it was also seen that the hole, which originally was round, had become pear-shaped, the part at C, fig. 5, which is a vertical section through the line N M, in fig. 4, which bore upon the pin, and consequently was in a state of compression, being considerably thickened; while, on the contrary, the other portion of the iron, at D and D, fig. 4, being under tension, had thinned by stretching, as shown in the section, through S T, fig. 6, so as to cause a tearing action to take place, which, having once commenced, would obviously, by the continuance of the same strain, rend through the entire head, no matter what its width might be. The first step in obtaining true proportions must, therefore, be to provide sufficient bearing area in the semi-cylindrical surface of the link (which we may call S), in order that it may not yield under the pressure of the pin until the head, at D, D, and the link at A, are simultaneously at the point of tearing asunder. Now, in wrought iron, the initial forces necessary to diminish the length of a bar by compression, or lengthen it by tension, are practically the same; and hence the surface S and the transverse area at A must be equal.

Fig. 4.

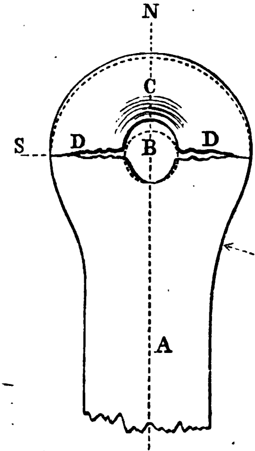
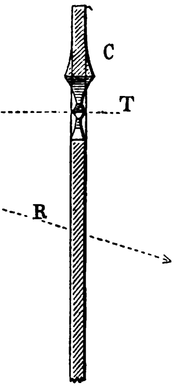


Fig. 5.



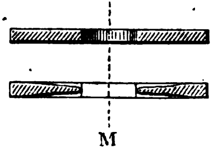


Fig. 6.

Two methods of effecting this present themselves—one by thickening the metal in the head, the other by enlarging the holes, and consequently the pin. The former would be inadmissible from the additional space it would occupy in the width of the bridge. In fig. 2 the surface S is 7.07, and the area at A, 10.25 square inches. Each link-head would, therefore, have to be widened 0.45 inch, and an arrangement of the links, such as that shown in page 40, would have to be 9¼ inches wider; the same addition being required at the other chain, would, together, amount to 18½ inches, which must be deducted from the available width of the roadway at the centre of the span, and would require increased width of rollers and bed-plates, &c., on the tower and anchorage; consequently, the requisite surface must be obtained by enlarging the diameter of the eye and of the pin.

Acordingly, a link was prepared, as shown in fig. 3, in which the hole was enlarged to 6 inches in diameter, everything else being the same as in fig. 1, except that the part D, D was now only 5¼ inches, instead of 6 inches in width, by the removal of the annular piece, the material in the head being diminished to that extent, instead of increased, as in fig. 2. The surface, S, was consequently augmented from 7.07 to 9.4 square inches. This link, so altered, yielded by a force of nearly 240 tons, which, compared with the 180 tons of the former experiments, is in the ratio of the area of the two surfaces S bearing the compression of the pin. The author adds: "From subsequent experiments, it has become evident that, had the pins of these chains been increased to 6½ inches diameter, giving a bearing surface of 10.2 square inches, the proper proportion between them and the body of the links would have been very nearly arrived at, while with those of 6 inches diameter about an inch of the body of the links was wasted." The sections of the shanks of the links were identical in each experiment, and are given below the drawings of the heads—the part which is hatched with diagonal lines—indicating the part which, at 27 tons the square inch, would be sufficient to give a resistance equal to that with which the heads were destroyed. In figs. 1 and 2 it amounts only to two-thirds, and in fig. 3 to nine-tenths of the whole. If, then, we assume that, of the 1600 tons already mentioned, about 1450 tons were applied to the shanks of the bar links, the remainder constituting the heads, we see that about 480 tons would have been not only useless but detrimental, as forming a permanent load on the structure, to the strength of which it added nothing. In fig. 3, only about 150 tons, out of 1450, are thus lost. With a pin, 6½ inches diameter, it is probable that the whole area, 10¼ square inches, would have been efficient.

These experiments, which teach that, under certain circumstances, greater strength may be obtained by diminishing the quantity of material employed when its distribution is altered, are illustrated by other instances, as in hydraulic presses, in the construction of artillery, and in screwed bolts, &c. (*Vide* E. Clarke, "Brit. Bridges," p. 609, &c.; Bramwell, "Report British Association," 1869.)

Finally, this simple rule was deduced: "The connecting pin, and the circular hole to receive it, to have a diameter equal to two-thirds of the width of the body or shank of the link. And, as the strain upon the metal in the head of the link is less direct or uniform than in the body, let the sum of the widths on each side of the pin (D and D) be made 10 per cent. greater than that of the body itself."

The author also suggested that, as the pins, if solid, would be of much larger section than is necessary to resist the action of shearing, a considerable saving in weight would be effected by having them made hollow and of steel.

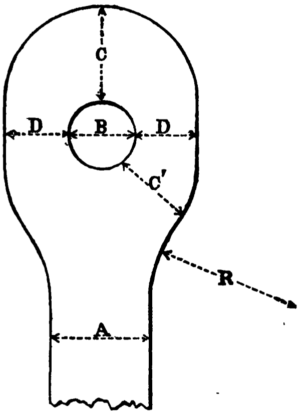


Fig. 7.

Mr. Berkeley, in the paper already referred to, gives the result of numerous experiments carried on for many years in order to arrive at the best proportions for the tension bars for Warren girders, to be erected on East Indian railways. Between twenty and thirty thousand such have been sent out to that country, where this construction of girder, as requiring but little skilled labour in finally erecting, is chiefly used, the tension bars, formed like those of suspension bridges, being fixed in their respective places by pins passing through eyes in the ends of flat links. The standard proportions he was led to adopt are shown in fig. 7, in which the width of the body of the link at A is taken as 100, the other parts as below:—

A — 100
B — 75
D — 60
C — 80
R — 500

These results point in the same direction as those of Sir C. Fox—namely, that hitherto the practice had been to make the pin too small in diameter, proportioning it rather with a view to its shearing resistance than to its proper bearing area. It may be doubted, however, whether it is needful to increase it beyond two-thirds of A. No practical increase of pin would give a uniform tension in the horizontal section, at D, D, though the difference between the extreme rate and the mean is less, as it is increased in relation to the whole width of the head. Again, the whole of the upper semi-cylindrical surface, S, cannot well be conceived to resist, at a uniform rate, the compression of the pin. Therefore, the area, S, might well be greater than that at A. When the diameter of the pin B is $\frac{7}{11}$ths of the breath at A (or 63·6, instead of 75 as above) the surface S and the transvere area at A are equal.

In many other details of structures we find the utility of these results. The horizontal tension member of bow-string girders has been sometimes formed of bar-links. Pitch-chains, and the links attached to the buckets of dredging-machines, are examples of material connected together, and strained in a similar manner by pins through eyes.

In some bridges the hole for the pin is made oval (in the direction of N M, fitting truly into the line S T, fig. 4) for facility in getting the main chains together, and it has been suggested that the form given by tangents to the eye meeting in a V in the line N M on the side towards A would be advantageous by transmitting the tensile strain from D and D to A in a more direct manner.

In the discussion which took place on the reading of the paper by Mr. Howard, (Vol. viii., p. 273, Minutes of Proceedings Inst. Civ. Engrs.), on rolling the links of suspension bridges without any welding, will be found some useful remarks. The paper itself contains an interesting account of the process, and the machinery employed.

The wood-cuts on the opposite page represent the form of the ends of tension bars of Warren girders of large span, and also their transverse sections taken from recent publications. The diameter of the eye in each is evidently very much out of proportion to the breadth of the body of the link. If the pin and the eye be properly proportioned to the strains in the part of the girder in which they are placed, then the breadth of the body of the bar is greatly in excess. In the upper bar, the eye of which is 2½ inches in diameter, the breadth should, by the above rule, be 3¾ inches instead of 6½;

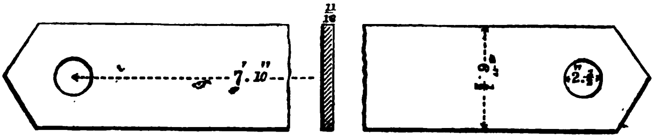


Fig. 8.

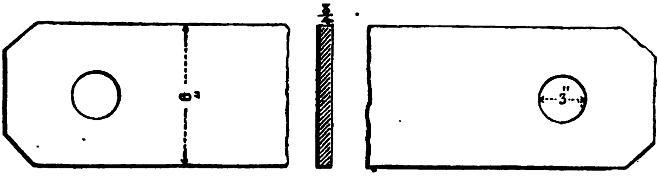


Fig. 9.

and in the other for an eye 3 inches in diameter, the bar should be 4½ instead of 9 inches. On the other hand, if the breadths of the respective bars be duly proportioned to their tensile strains, the diameters of the pins are quite too small. If, however, we compare the transverse area of the bars with the transverse area of the pins, we find they are very nearly equal, indicating perhaps the faulty principle on which they have been designed. The transverse area of the pin (to resist shearing) is, in the upper drawing, 4.9 and the link 4.47 square inches. In the lower, the pin is 7, and the link 6.75 square inches.

The wood-cuts in the following page show the construction and proportions of the links of pitch chain which afford a good example of wrought iron united by pins and links, double and single alternately, which being perfectly free at the joints is capable of being wound round the sheave of an iron block; it is also used as an endless chain driven by cogs which project between the double link and press on the head of the single one.

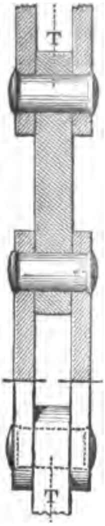


Fig. 10.

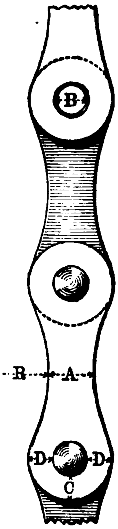


Fig. 11.

The left-hand drawing shows, down to the full line, a vertical section through the axes of the pins which are shown in full with their countersink and rivet heads. The lower part is an end view. The resultant force passes through the line TT.

The right-hand drawing is an elevation of two links of a chain, the shaded portions being the single links; at B the pin is removed, and the vertical projection of the countersink shown. The sides of all the links must be carefully turned to the same radius as the sheave or the barrel of the winding drum. In this drawing if (using the same notation as in page 335), A = 100, then B = 75, D = 52, C = 52, and R = 410·5. The pitch, or distance from centre to centre, being about 383. In dredging machines the upper tumbler which drives all the buckets is square, and the links, which carry the buckets, are made quite straight on the side which is in contact with it and in effective length equal to the side of the square tumbler.

No. II.

The following Tables (pp. 340–343) have been greatly extended from those given in the "Philosophical Transactions," by Mr. E. Hodgkinson. That of the 3.5th powers of the diameters contains every tenth and eighth of an inch, from 1 to 15 inches inclusive, and from 16 to 19.9 inches, every tenth and every quarter of an inch. That of the 1.63 powers of the length extends from 1 to 40 feet, giving every intermediate 6 inches, from 40 to 50 every foot of length, and from 50 to 80 every fifth foot; and also a logarithm, which is that of the quotient of the constant 42.35, divided by the length raised to the 1.63 power.

To apply these Tables, let it be required to find the ultimate strength of a column, 24 feet 6 inches in height, and 10 inches external diameter, the metal being $\frac{7}{8}$-inch thick, which gives an internal diameter of $8\frac{1}{4}$ inches. The strength is therefore,

$$42.35 \times \frac{10^{3.5} - 8.25^{3.5}}{24.5^{1.63}}.$$

From the Table, the value of the numerator is

$$3162.28 - 61.89 = 3100.39;$$

and if we do not use the logarithm opposite, 24.5, in the Table of the powers of the length, we must multiply by the constant, and divide by 183.81, which is the number opposite to 24.5, giving (3100.39 × 42.35 =) 131301.5 ÷ 183.81 = 714.33 tons.

In the other case we have but to add to the log. of 3100.39, which is

$$\begin{array}{lr} & 3.49142 \\ \text{To that in the Table of lengths,} & \underline{\bar{1}.36248} \\ & 2.85390 \end{array}$$

which answers to the number 714.33, as before, but with greatly reduced labour of computation.

If the coefficient of stability be determined at 8, then the proper load of this column would be 89.3 tons.

Again, let a column be designed 50 feet in height, and 19.90 inches external diameter, and having a thickness of $1\frac{1}{8}$th inch of metal, giving an internal diameter of 17.65 inches, for the 3.5^{th} powers of this we may take the arithmetic mean between the 3.5^{th} powers of 17.60 and 17.70, being the next lower and higher numbers given in the Table, namely—

$$\begin{array}{lr} \text{23100.52, and to its log.} & 4.36362 \\ \text{Add that opposite 50 . . .} & \underline{\bar{2}.85753} \\ \text{Answering to 1664 tons . .} & 3.22115 \end{array}$$

TABLE *of the 3.5th Powers of Numbers, being the Diameters of Columns in Inches, with the Tenths and Eighths, up to 15, in most of them.*

Diameters.	3·5th Powers.	Diameters.	3·5th Powers.	Diameters.	3·5th Powers.
1.000	1.000	4.000	128.00	7.000	907.49
.100	1.396	.100	139.55	.100	953.68
.125	1.510	.125	142.55	.125	965.49
.200	1.893	.200	151.83	.200	1001.53
.250	2.184	.250	158.26	.250	1026.10
.300	2.505	.300	164.87	.300	1051.07
.375	3.048	.375	175.16	.375	1089.35
.400	3.247	.400	178.68	.400	1102.33
.500	4.1335	.500	193.30	.500	1155.35
.600	5.181	.600	208.76	.600	1210.17
.625	5.470	.625	212.76	.625	1224.11
.700	6.406	.700	225.08	.700	1266.83
.750	7.0898	.750	233.57	.750	1295.85
.800	7.824	.800	242.29	.800	1325.35
.875	9.026	.875	255.80	.875	1370.50
.900	9.454	.900	260.43	.900	1385.78
2.000	11.314	5.000	279.51	8.000	1448.15
.100	13.4205	.100	299.57	.100	1512.51
.125	13.99	.125	304.73	.125	1528.87
.200	15.7935	.200	320.63	.200	1578.83
.250	17.086	.250	331.56	.250	1612.83
.300	18.452	.300	342.74	.300	1647 33
.375	20.64	.375	360.02	.375	1699 99
.400	21.416	.400	365.91	.400	1717.83
.500	24.705	.500	390.18	.500	1790.47
.600	28.34	.600	415.58	.600	1865.30
.625	29.31	.625	422.10	.625	1884.34
.700	32.34	.700	442.14	.700	1942.31
.750	34.49	.750	455.87	.750	1981.66
.800	36.73	.800	469.89	.800	2021.53
.875	40.29	.875	491.51	.875	2082.55
.900	41.53	.900	498.86	.900	2103.12
3.000	46.76	6.000	529.09	9.000	2187.00
.100	52.45	.100	560.60	.100	2273.24
.125	53.95	.125	568.70	.125	2295.12
.200	58.62	.200	593.43	.200	2361.88
.250	61.89	.250	610.35	.250	2407.11
.300	65.28	.300	627.61	.300	2452.96
.375	70.62	.375	654.15	.375	2522.87
.400	72.47	.400	663.18	.400	2546.52
.500	80.21	.500	700.16	.500	2642.61
.600	88.52	.600	738.59	.600	2741.25
.625	90.69	.625	748.45	.625	2766.30
.700	97.43	.700	778.51	.700	2842.50
.750	102.12	.750	799.03	.750	2894.12
.800	106.96	.800	819.94	.800	2946.40
.875	114.54	.875	851.01	.875	3026.10
.900	117.15	.900	862.92	.900	3052.98

TABLE *of the* 3.5*th Powers of Numbers, being the Diameters of Columns in Inches, with the Tenths and Eighths, up to* 15, *in most of them*—(continued).

Diameters.	3·5th Powers.	Diameters.	3·5th Powers.	Diameters.	3·5th Powers.
10.000	3162.28	**13**.000	7921.40	**16**.00	16384.00
.100	3274.35	.100	8136.72	.10	16745.20
.125	3302.80	.125	8191.20	.20	17112.06
.200	3389.23	·200	8356.20	.25	17297.60
.250	3447.73	.250	8467.51	.30	17484.60
.300	3506.95	.300	8579.87	.40	17862.90
.375	3597.15	.375	8750.41	.50	18247.10
.400	3627.57	.400	8807.79	.60	18637.10
.500	3751.13	.500	9040.0	.70	18033.00
.600	3877.67	.600	9276.54	.75	19233.20
.625	3909.77	.625	9336.36	.80	19434.90
.700	4007.22	.700	9517.48	.90	19842.80
.750	4073.14	.750	9639.61		
.800	4139.84	.800	9762.85	**17**.00	20256.80
.875	4241.36	.875	9949.83	.10	20676.90
.900	4275.56	.900	10012.70	.20	21103.25
				.25	21318.75
11.000	4414.43	**14**.000	10267.10	.30	21535.80
.100	4556.49	.100	10526.08	.40	21974.70
.125	4592.51	.125	10591.55	.50	22419.85
.200	4701.79	.200	10789.70	.60	22871.50
.250	4775.66	.250	10923.25	.70	23329.55
.300	4850.36	.300	11058.00	.75	23561.04
.375	4963.98	.375	11262.30	.80	23794.15
.400	5002.27	.400	11331.00	.90	24265.30
.500	5157.54	.500	11608.80	**18**.00	24743.10
.600	5316.22	.600	11891.45	.10	25227.55
.625	5356.43	.625	11962.85	.20	25718.75
.700	5478.36	.700	12178.94	.25	25966.90
.750	5560.74	.750	12324.60	.30	26216.75
.800	5644.00	.800	12471.43	.40	26721.60
.875	5770.56	.875	12694.03	.50	27233.35
.900	5813.19	.900	12768.86	.60	27752.05
				.70	28277.80
12.000	5985.96	**15**.000	13071.30	.75	28543.32
.100	6162.39	.100	13378.86	.80	28810.60
.125	6207.06	.125	13456.55	.90	29350.55
.200	6342.48	.200	13691.55		
·250	6433.93	.250	13849.80	**19**.00	29897.70
.300	6526.31	.300	14009.40	.10	30452.08
.375	6666.66	.375	14251.25	.20	31013.75
.400	6713.92	.400	14332.50	.25	31297.30
.500	6905.34	.500	14660.90	.30	31582.80
.600	7100.63	.600	14994.65	.40	32159.26
.625	7150.06	.625	15078.90	.50	32743.20
.700	7299.83	.700	15333.80	.60	33334.68
.750	7400.92	.750	15505.36	.70	33933.70
.800	7503.00	.800	15678.34	.75	34236.15
.875	7658.00	.875	15940.35	.80	34540.47
.900	7710.16	.900	16028.40	.90	35154.90

TABLE *of Numbers raised to the* 1.63 *Power, being the Lengths in Feet, of Columns, and also the Logarithm of the Constant* 42.35, *divided by the several Powers.*

Length in Feet.	1·63 Powers of Length.	Log. of Constant, divided by Column 2.	Length in Feet.	1·63 Powers of Length.	Log. of Constant, divided by Column 3.
1.0	1.0	1.62685	19.0	121.44	$\bar{1}$.54249
2.0	3.10	1.13549	19.5	126.69	$\bar{1}$.52410
2.5	4.45	0.97823	20.0	132.03	$\bar{1}$.50618
3.0	5.99	0.84914	20.5	137.45	$\bar{1}$.48870
4.0	9.58	0.64549	21.0	142.96	$\bar{1}$.47164
5.0	13.78	0.48753	21.5	148.55	$\bar{1}$.45498
5.5	16.10	0.42006	22.0	154.22	$\bar{1}$.43871
6.0	18 55	0.35846	22.5	160.00	$\bar{1}$.42279
6.5	21.14	0.30180	23.0	165.81	$\bar{1}$.40724
7.0	23.85	0.24936	23.5	171.73	$\bar{1}$.39200
7.5	26.69	0.20050	24.0	177.72	$\bar{1}$.37711
8.0	29.65	0.15481	24.5	183 81	$\bar{1}$.36248
8.5	32.73	0.11190	25.0	189.95	$\bar{1}$.34821
9.0	35.93	0.07139	25.5	196.18	$\bar{1}$.33419
9.5	39.24	0.03316	26.0	202.49	$\bar{1}$.32045
10.0	42.66	$\bar{1}$.99683	26.5	208.88	$\bar{1}$.30696
10.5	46.20	$\bar{1}$.96231	27.0	215.34	$\bar{1}$.29373
11.0	49.83	$\bar{1}$.92938	27.5	221.87	$\bar{1}$.28074
11.5	53.57	$\bar{1}$.89792	28.0	228.49	$\bar{1}$.26799
12.0	57.42	$\bar{1}$.86779	28.5	235.17	$\bar{1}$.25546
12.5	61.37	$\bar{1}$.83889	29.0	241.94	$\bar{1}$.24314
13.0	65.42	$\bar{1}$.81114	29.5	248.77	$\bar{1}$.23105
13.5	69.57	$\bar{1}$.78443	30.0	255.68	$\bar{1}$.21915
14.0	73.82	$\bar{1}$.75866	30.5	262.67	$\bar{1}$.20744
14.5	78.17	$\bar{1}$.73382	31.0	269.72	$\bar{1}$.19593
15.0	82.61	$\bar{1}$.70982	31.5	276 85	$\bar{1}$.18461
15.5	87.14	$\bar{1}$.68663	32.0	284.85	$\bar{1}$ 17346
16.0	91.77	$\bar{1}$.66415	32.5	291.32	$\bar{1}$.16249
16.5	96.49	$\bar{1}$.64237	33.0	298.66	$\bar{1}$.15167
17.0	101.31	$\bar{1}$.62122	33.5	306.07	$\bar{1}$.14104
17.5	106 21	1.60070	34.0	313.55	$\bar{1}$.13054
18.0	111.20	$\bar{1}$.58075			
18.5	116.28	$\bar{1}$.56136			

TABLE *of Numbers raised to the* 1.63 *Power*—(continued).

Length in Feet.	1·63 Powers of Length.	Log. of Constant, divided by Column 2.	Length in Feet.	1·63 Powers of Length.	Log. of Constant, divided by Column 3.
34.5	321.10	$\bar{1}$.12021	43.0	459.79	$\bar{2}$.96430
35.0	328.72	$\bar{1}$.11002	44.0	477.34	$\bar{2}$.94803
35.5	336.41	$\bar{1}$.09998	45.0	495.14	$\bar{2}$.93212
36.0	344.17	$\bar{1}$.09008	46.0	513.21	$\bar{2}$.91655
36.5	351.99	$\bar{1}$.08032	47.0	531.52	$\bar{2}$.90133
37.0	359.89	$\bar{1}$.07068	48.0	550.07	$\bar{2}$.88643
37.5	367.85	$\bar{1}$.06118	49.0	568.89	$\bar{2}$.87183
38.0	375.88	$\bar{1}$.05181	50.0	587.92	$\bar{2}$.85753
38.5	383.97	$\bar{1}$.04255	55.0	686.73	$\bar{2}$.79007
39.0	392.13	$\bar{1}$.03342	60.0	791.38	2.72847
39.5	400.37	$\bar{1}$.02439	65.0	901.66	$\bar{2}$.67181
40.0	408.66	$\bar{1}$.01549	70.0	1017.45	$\bar{2}$.61934
40.5	417.02	$\bar{1}$.00669	75.0	1138.54	$\bar{2}$.57050
41.0	425.44	$\bar{2}$.99802	80.0	1264.84	$\bar{2}$.52482
42.0	442.48	$\bar{2}$.98096			

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